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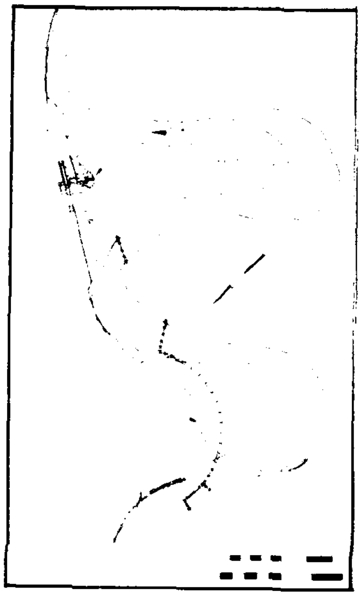
RED RIVER WATERWAY, LOCK AND DAM NO. 4

Report 3
SEDIMENTATION CONDITIONS

Hydraulic Model Study



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13. ABSTRACT *(Maximum 200 words)*

A movable-bed physical model was used to investigate and solve potential channel development and maintenance problems associated with Lock and Dam No. 4 on the Red River in Louisiana. The model reproduced the Red River from 1967 river mile 213.1 to 204.7 using a distorted scale of 1:120 horizontally and 1:80 vertically. The model was adjusted to reproduce prototype conditions before the inclusion of the proposed project. The upstream approach to Lock and Dam No. 4 will include a short-radius man-made cutoff through the neck of the existing bend, shortening the length of the existing bendway by approximately 1 mile. The lock and dam will also be located in an excavated channel, cutting off about 3 miles of an existing bendway. Procedures were developed to properly model channel modifications that significantly shorten the channel length. Various configurations of dikes and berms were evaluated to determine the optimum configuration for maintaining a satisfactory navigation channel.

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PREFACE

On 13 August 1968, the 90th Congress approved Public Law 90-483 authorizing the construction of the Red River Waterway, Louisiana, Texas, Arkansas, and Oklahoma Project in accordance with the recommendations of the Chief of Engineers as contained in House Document No. 304, 90th Congress, 2nd Session. The model investigation reported herein was requested by the US Army Engineer District, New Orleans (LMN), in a letter dated 15 November 1976 to the US Army Engineer Waterways Experiment Station (WES), subject: Red River Waterway, Lock and Dam No. 4, Model Study.

The River Regulation Branch of the Waterways Division, Hydraulics Laboratory, WES, began model design and construction in January 1979. The investigation was suspended in August 1979, due to the relocation of the structure in the reformulation of the Red River Project. Work on the new design began in May 1980, with construction of the model beginning in November 1980. This work was suspended per a letter, dated 3 June 1981, from LMN requesting a shift in priority to other US Army Engineer Division, Lower Mississippi Valley (LMV), studies. During this period of model inactivity, the responsibility for the Red River Project was transferred from LMN to the US Army Engineer District, Vicksburg (LMK). Model design and construction were resumed in October 1984 and completed in November 1984. Model adjustment and verification were then initiated, but stopped in February 1985 to shift personnel to expedite the Lock and Dam No. 3, Red River, model study, as requested by LMK. In January 1986, model adjustment resumed but was again suspended in March 1986 due to priority established by LMK for the Lock and Dam No. 2 and 3, Red River, model studies. In July 1986, due to the work load in the River Regulation Branch, the model was transferred to the Potamology Branch of the Waterways Division. Model reactivation was completed in October 1986 and model adjustment was initiated. The model adjustment was completed in April 1987. The base test was completed in June 1987. Modifications to the model to include the lock and dam and cutoffs were completed in September 1987 and testing was initiated. The testing program consisted of eight plans and was completed in April 1988. LMV and LMK were informed of the progress of the study through progress reports and periodic transmittal of preliminary results. In addition, representatives from LMK visited WES during the course of the study to observe the tests and discuss test results.

In addition to the hydraulic movable-bed model study, two physical model studies and two numerical model studies were conducted at WES. The additional studies included a fixed-bed navigation model study (Report 2); a hydraulic structures model study (Report 4); a numerical model sedimentation study of upstream and downstream approaches to Lock and Dam No. 4 (Report 5); and a numerical model sedimentation study of the Red River upstream and downstream of Lock and Dam No. 4 (Report 6). This is Report 3 of the series. Report 1, to be published later, will summarize all of the model studies.

The work was conducted under the general supervision of Messrs. H. B. Simmons, former Chief of the Hydraulics Laboratory (retired); F. A. Herrmann, Jr., present Chief of the Hydraulics Laboratory; and R. A. Sager, Assistant Chief of the Hydraulics Laboratory; and under the direct supervision of Messrs. J. E. Glover, former Chief of the Waterways Division (retired); M. B. Boyd, present Chief of the Waterways Division; T. J. Pokrefke, acting Assistant Division Chief; and C. R. Nickles, Acting Chief of the Potamology Branch. The engineer in immediate charge of the model was Mr. D. S. Mueller, assisted by Mr. D. M. Maggio and Ms. K. Anderson-Smith, all of the Potamology Branch, and Mr. M. E. Caldwell, Navigation Branch, Waterways Division. This report was prepared by Messrs. Mueller, Maggio, and Pokrefke. Representatives of LMK who were actively involved in the study were Messrs. Basil Arthur and Terry Smith. This report was edited by Mrs. M. C. Gay, Information Technology Laboratory, WES.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609344	kilometres

RED RIVER WATERWAY, LOCK AND DAM NO. 4

SEDIMENTATION CONDITIONS

Hydraulic Model Study

PART I: INTRODUCTION

Location and Description of Prototype

1. The Red River (Figure 1) flows easterly from the northwest portion of Texas along the border between Texas and Oklahoma into southwestern Arkansas where it turns southerly into northwestern Louisiana to Shreveport, LA, then southeasterly through northern Louisiana to Alexandria, LA, then easterly to join the Old River and form the Atchafalaya River. The Atchafalaya River flows through the southeastern portion of Louisiana to the Gulf of Mexico downstream of Morgan City, LA.

2. The Red River is characterized by large fluctuations in stage and discharge and shifting bed and banks. The bed and banks of the Red River are composed of highly variable combinations of silts, clays, and sands. Long periods of low flow, narrow bends of short radii, and a heavy sediment load have limited the use of the Red River for movement of cargo by barges.

Plan of Development

3. The 90th Congress authorized the development of the Red River Waterway with the passage of Public Law 90-483 on 13 August 1968. As presently authorized, the project provides for the improvement of the Red River and its tributaries into Louisiana, Arkansas, Texas, and Oklahoma through coordinated development for navigation, bank stabilization, flood control, recreation, fish and wildlife, and water-quality control. The primary function of the project is to establish a 9-ft*-deep by 200-ft-wide navigation channel from Old River to Lake of Pines near Daingerfield, TX, by a system of nine locks and dams, extensive channel realignment, a number of cutoffs, and miles of

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

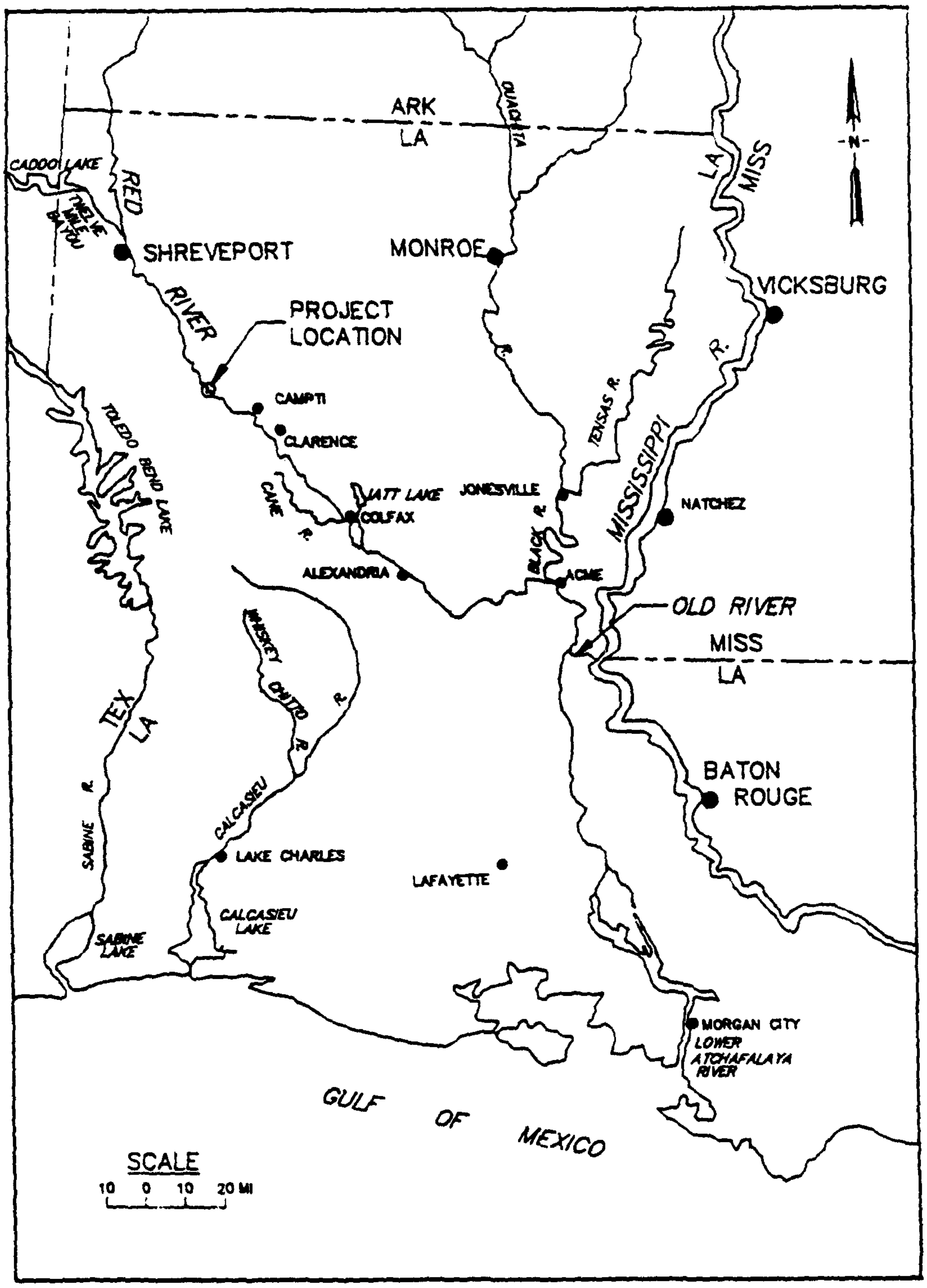

Figure 1. Location map

channel training and stabilization works. The project consists of four distinct reaches: (a) Mississippi River to Shreveport, LA; (b) Shreveport to Daingerfield, TX; (c) Shreveport to Index, AR; and (d) Index to Denison Dam, TX. The Appropriation Act of 1971, approved 7 October 1970 as Public Law 91-439, provided authority to initiate preconstruction planning in the Mississippi River to Shreveport Reach, the only reach pertinent to this study.

4. The Mississippi River to Shreveport reach of the project is located along the Red River in central and northwestern Louisiana. Within this reach, the project plan will establish a navigable channel approximately 236 miles long and 9 ft deep by 200 ft wide from the Mississippi River to Shreveport via the Old and Red Rivers. Five locks and dams will furnish the required maximum lift of 141 ft. The locks are to be 84 ft wide with a usable length of 685 ft. Bank revetment and other complementary stabilization and river training works will be used to stabilize the route of the newly developed channel.

Lock and Dam No. 4

5. Lock and Dam No. 4 is the fourth lock and dam on the Mississippi River to Shreveport reach of the Red River Waterway. The proposed lock and dam will be located in a cutoff between 1967 river miles 205 and 210 (Plate 1) and have a normal pool at el 120.* The plan herein consists of a lock with upstream and downstream guard walls, a tainter-gated spillway, and a hinged crest-gated spillway. In addition, the natural channel upstream and downstream of Lock and Dam No. 4 will be realigned to facilitate better navigation to the structure.

Need for and Purpose of Model Study

6. The development of the Red River for navigation will require the solution to many channel development and maintenance problems. Solutions to these problems on a river transporting large amounts of sediment are very complex and interdependent. The upstream approach to Lock and Dam No. 4 will include a short-radius man-made cutoff through the neck of the existing bend,

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* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

shortening the length of the existing bendway by approximately 1 mile (Plate 1). The actual lock and dam structure will also be constructed in an excavated channel, cutting off about 3 miles of an existing bendway. Therefore, the navigation plan will shorten this reach of river approximately 4 miles, which could cause sedimentation problems. Because the navigation plan differed significantly from the existing conditions, it was unlikely that the existing flow conditions and channel configuration could be used to anticipate the postproject flow conditions and channel configuration. Therefore, it was decided that a hydraulic movable-bed model would be used to evaluate and determine the flow conditions and channel configuration of the proposed project. The purposes of the model study were as follows:

a. To evaluate the proposed channel alignment and the arrangement of the lock and dam.

b. To study tendencies for scour and fill in the approaches to the lock and dam.

c. To design and test training structures necessary to provide acceptable navigation conditions and minimize dredging requirements and scour problems.

Part II: THE MODEL

Description

7. This study used a movable-bed model constructed to a distorted scale of 1:120 horizontally and 1:80 vertically. The model reproduced a reach of the Red River between 1967 river miles 213.1 and 204.7 (Plate 1). A supplemental slope of 1 ft/mile was added to the average natural slope (0.5 ft/mile) of the overbank areas for a total slope of overbank areas in the model of 1.5 ft/mile. The overbank areas were composed of molded concrete and crushed stone with a light coating of cement. Revetted bank lines, dikes, and bed protection were all reproduced using crushed stone with a light coating of cement. The bed of the model was molded in crushed coal having the following properties: $d_{84} = 5.5$ mm, $d_{50} = 2.9$ mm, $d_{16} = 1.5$ mm, and a specific gravity of 1.30.

8. Several sources of information were used to obtain sufficient detail to reproduce the topographic and geometric features of this reach. The overbank portion of the model was constructed in accordance with a combination of contours and elevations shown on the Red River, Louisiana, Hydrographic Survey of April 1978, and from the US Geological Survey (USGS) quadrangle sheets. The channel portion of the model was molded in accordance with a composite 1967 and 1978 hydrographic survey supplied by US Army Engineer District, New Orleans, with the area between 1967 river mile 210.4 and 212.2 taken from the 1967 survey, and the remaining portion of the reach being made up of contours taken from the 1978 hydrographic survey. Since the majority of this survey was taken from the 1978 hydrographic survey, this survey will be referred to as the 1978 prototype survey for the remainder of the report.

9. The two planned cutoffs and the lock and dam were installed in accordance with plans furnished by US Army Engineer District, Vicksburg. The overbank areas in these cutoffs were composed of crushed stone with a light coating of cement and were molded using the supplemental slope determined during adjustment. The structures, including the lock and dam and overflow sections, were constructed of sheet metal and plastic. Lock and dam gates were simulated with simple sheet metal slide-type gates.

Appurtenances

10. Water was supplied to the model by a 10-cfs centrifugal flow pump operating in a recirculating system. The inflow was measured at the upstream end of the model by a 6- by 3-in. venturi meter to provide accurate measurement of flow over the range of discharges necessary to reproduce the adjustment and typical hydrographs. During the adjustment to existing conditions, water-surface elevations were measured using a single point gage and 11 piezometers located in the model channel and connected to stilling basins in a centrally located gage pit. Water-surface elevations were controlled with an adjustable tailgate at the downstream end of the model. During testing of the planned channel realignment, water-surface elevations were measured by 12 point gages located at various points along the channel. Water-surface elevations upstream of the dam were controlled by adjusting the gates in the dam. Water-surface elevations downstream of the dam were controlled by the adjustable tailgate at the downstream end of the model.

11. Bed material was introduced into the flow at the upstream end of the model using a graduated container to measure the volume of bed material introduced. A sediment trap was provided at the downstream end of the model where discharged material could accumulate. The volume of material discharged for any specific period was measured using a graduated container.

12. Bed elevations in the model were obtained using a surveying rod that permitted the reading of elevations in prototype feet. Sheet metal templates were used for molding the model bed to the 1978 prototype survey. A carefully graded rail was installed along each side of the channel to (a) adjust the supplemental slope of the bed; (b) support the templates at the correct elevation; (c) support a rail used to survey the model bed; and (d) provide vertical control for installing structures in the model.

Model Adjustment

Description

13. Before proposed plans can be tested, adjustments to a model must be made to ensure that the model reproduces, to a reasonable degree of accuracy, changes and trends that have been observed in the prototype. This process is referred to as model adjustment. The adjustment process establishes the

discharge scales, quantity of bed material introduced, supplemental slope, and model operating procedures necessary to obtain proper movement of the bed material so that the model reproduces prototype conditions.

Conventional procedure

14. An ideal adjustment of a movable-bed model requires two prototype bed surveys about 1 year apart and stage and discharge hydrographs for that location and period of time. The hydrographs used should be neither high-water nor low-water hydrographs but cover the range of stages and discharges typical of the study reach. The model bed is molded to the first prototype survey. The flows of the hydrograph that occurred between the surveys are introduced at the upper end of the model and the water surface is controlled at the downstream end of the model so that prototype stages are reproduced at the center of the model. Bed material is introduced at the upper end of the model at a rate that causes no more scour or deposition at the upstream end of the model than occurred in the prototype for the same period of time. At the end of the hydrograph, the model bed is surveyed and the bed configurations are compared with those from the second prototype survey. If the model does not reproduce the prototype survey to the desired accuracy, modifications to the supplemental slope, discharge scales, the rate of introduction of bed material, and model operating procedures are made and the process repeated. This sequence is repeated until the model satisfactorily reproduces the prototype bed configurations. Once the model has been adjusted, the scales and procedures are fixed and used in the testing program.

Actual procedure used

15. Stages and discharges at 1967 river mile 209, which is located at the center of the study reach, were required for the model adjustment. There was no gaging station in the study reach and therefore hydrographs and rating curves had to be interpolated from gaging stations outside the study reach. Discharges were interpolated using the Shreveport (river mile 277.9) and Alexandria (river mile 104.9) discharge data. The discharge at river mile 209 was computed based on the ratio of drainage areas as follows:

$$Q_{209} = Q_{Shrev} + (Q_{Alex} - Q_{Shrev})\left(\frac{D_{A_{Shrev-209}}}{D_{A_{Shrev-Alex}}}\right) \qquad (1)$$

where Q is the discharge at the respective location and D_A is the drainage area between the two specified locations. Prototype stages at river mile 209 were linearly interpolated between the gages at Coushatta, LA (river mile 220.6), and Grand Ecore, LA (river mile 183.9). There were gages at Campti, LA (river miles 193.9 and 192.9), but the data for these gages appeared to be in error and were not used. The HEC-2 model originally provided by the Vicksburg District did not produce rating curves that compared favorably with prototype data interpolated using the procedure outlined herein (Plate 2). A new HEC-2 model was developed by US Army Engineer Waterways Experiment Station (WES) personnel using all the parameters in the original model but using cross sections every 0.1 mile rather than every 2 miles as in the original model. This new HEC-2 model produced rating curves that compa d favorably with the prototype data (Plate 2). Therefore, this new HEC-2 model was used to develop the rating curve at river mile 209 that was used in this study.

16. Two prototype surveys were available for the test reach, a 1978 hydrographic survey (Plate 3) and a 1981 hydrographic survey (Plate 4). The 1981 survey was actually taken over a period from 1980 through 1981 and was not suitable for adjusting the model for two reasons. The model had been constructed prior to the completion of the 1981 survey and thus contained the bank lines shown on the 1978 prototype survey. Due to active bank erosion, the bank lines in the 1981 survey were significantly different from those of the 1978 survey in several places. For the period between June 1979 through May 1981, no discharge greater than 50,000 cfs was observed (Plate 5), which is not typical of the range of flows that normally occurred in the reach.

17. A long period of low flow prior to 1978, shown in Plate 5, made the 1978 prototype survey undesirable for adjustment as well. However, for the model to be of value, it was essential that it be adjusted to reproduce channel configurations generally representative of those expected in this reach of the Red River under similar circumstances. Analysis of the discharge hydrographs occurring before and after 1978 showed that the hydrograph for the period between September 1978 and August 1979 provided a range of flows typical of this reach of river and was thus used as the adjustment hydrograph (Plate 6). Therefore, the 1978 prototype survey was used as the beginning prototype survey to mold the model and as the ending prototype survey to

compare with the survey obtained from the model at the end of the adjustment hydrograph.

18. The adjustment was performed with the model initially molded to the 1978 prototype survey, and the model operated reproducing the hydrograph for the period September 1978 through August 1979. During the adjustment there was sufficient sediment transport to have caused a change in the bed configuration; however, the model ending survey compared favorably to the beginning prototype survey. The discharge ratio curve, sediment input curve, and supplemental slope for the final adjustment are shown in Appendix A. The results of the final adjustment run, displayed in Plate 7, compared to the 1978 prototype survey (Plate 3), show that the model reproduced the general characteristics of the prototype reach. To verify the final adjustment, the low flows from September 1979 through December 1980 were reproduced (Plate 8) and the ending survey (Plate 9) compared to the 1981 hydrographic survey. This comparison indicated that the model satisfactorily reproduced the general characteristics of the prototype. The alignment of the deepest part of the channel was generally the same in the model as in the prototype with bars and crossings in the proper places. The model channel was generally deeper than indicated by the prototype survey with the bar areas slightly higher, but to such a small extent that this adjustment was considered adequate for the purposes of this study.

PART III: TESTS AND RESULTS

Introduction

19. After adjustment and verification of the model were completed, tests for various plans were conducted to determine the channel development tendencies resulting from one or more reproductions of the typical hydrograph. During testing, channel improvements likely to affect the water level near the center of the model were added. Therefore, the water level control that reproduced prototype stages near the center of the model during adjustment was moved to the downstream end of the model for the base test and plan testing. Because prototype stages are not reproduced at the downstream end of a model, the prototype stages for the end of the model are converted to model stages r that location.

20. The typical hydrograph developed for Lock and Dam No. 4 was based on the discharge duration curve from Design Memorandum No. 29 (US Army Engineer District, Vicksburg, 1986) and analysis of approximately 25 years of hydrologic data. Based on this analysis the typical hydrograph (Plate 10) used for the Lock and Dam No. 4 model was similar in shape to the 1969 calendar year hydrograph.

21. Each reproduction of the typical hydrograph is herein referred to as a "run." Most of the tests of improvement plans or modifications were started with the bed molded to an initial condition (Plate 11) developed from the 1978 prototype survey and plans for the proposed cutoffs and improvements. When only the upstream or downstream section of the model was changed, often only that section was remolded. The bed was surveyed and mapped at the end of each run.

22. Normally a given improvement plan is tested until the bed configuration has reached an equilibrium condition and additional repetitions of the typical hydrograph do not change the bed. However, due to time constraints imposed by the design schedule of Lock and Dam No. 4, the various improvement plans were not run to equilibrium but were tested for several repetitions of the typical hydrograph, which was considered long enough to provide an indication of the tendencies of the bed configuration resulting from the plan.

Base Test

Description

23. A base test was conducted on the preproject channel configuration to provide an indication of the channel changes that would result from the typical hydrograph independent of the proposed channel improvements. The base test consisted of five reproductions of the typical hydrograph. Before the start of the first hydrograph reproduction, the model was molded to the 1978 prototype survey. Each succeeding reproduction of the hydrograph used the bed configuration of the previous run.

Results

24. Results of the base test, shown in Plate 12, display bed configurations similar to those of the adjustment and low-water verification surveys (Plates 7 and 9, respectively). The location and magnitude of areas below el 90 were very similar to the adjustment and low-water verification surveys. The typical hydrograph produced deeper scour at the outside of bends and higher point bars on the inside of bends than were observed on either of the other two model surveys. This behavior in bends could be expected as the typical hydrograph consisted of higher discharges than the hydrographs used for the adjustment and low-water verification. Crossing depths from the base test were very similar to those observed on the other surveys. The model produced a middle bar between ranges 5 and 10 to the right of the channel that was not observed in the adjustment and low-water verification surveys. Again this difference is likely the result of higher flows, which had sufficient depth over the bars to rearrange the bar sediments.

Test Procedure

Introduction

25. Techniques and procedures for modeling reaches that have been significantly shortened by cutoffs between the preproject and postproject conditions are not covered in Franco (1978). Lock and Dam Nos. 1 and 2 both required cutoffs in the prototype; however, the modeling techniques used for these reaches were not applicable at Lock and Dam No. 4. The model of Lock and Dam No. 1 did not reproduce the large bend that was cut off by the construction of the lock and dam. The upper and lower reaches were constructed and connected using a concrete trough to carry the flow during the adjustment

process. Lock and Dam No. 2 required a small cutoff that did not significantly shorten the reach of river. Therefore, techniques to model major cutoffs and locks and dams were developed for the study reported herein. These techniques should be generally applicable to all model studies of major cutoffs and locks and dams.

Changes in stage control

26. The addition of the lock and dam to the model required additional stage controls. The pool elevation had to be controlled at the upstream side of the dam to reproduce the anticipated prototype pool elevations. The tailwater normally measured just downstream of the dam also had to be maintained at the anticipated prototype stages. During the adjustment process, stages are reproduced on the gage near the center of the model. Due to distortion, this is the only gage on the model that reproduces prototype stages. Therefore, to control the model from a location other than the gage location near the center of the model, the prototype stages at the new location must be converted to model stages to maintain the same rating curves for all gages as when the gage near the center of the model was used for control. Because the dam was not located near the center of the model, a prototype-to-model stage relation curve, developed from model stages collected during the adjustment, was used to determine model stages for the new location of the control gages (Appendix B).

27. Any changes to the configuration of the exit channel influence the tailwater stages at the dam; therefore, the desired location of the tailwater control is at the downstream end of the model. The computed prototype stages for the gage location at the end of the physical model were interpolated from the HEC-2 model dated 22 January 1987 (Plate 13). This HEC-2 model was provided by the Vicksburg District to be used instead of the rating curves published in Design Memorandum No. 29 (US Army Engineer District, Vicksburg, 1986).

28. The location of the headwater control is in a cutoff that did not exist in the preproject configuration used for adjustment of the model. Therefore, no adjustment data for that location were available to develop a prototype-to-model stage relation curve. The cutoff conveys all of the water and has the same total drop in elevation as the longer natural channel. Therefore, if the stages at the beginning and end of the cutoff are the same as the natural channel, then the stage at the center of the cutoff should

correlate very closely with the stage at the center of the natural channel to be cut off. Based on this reasoning, the distance through the cutoff to the location of the headwater control was computed as a percentage of the length of the whole cutoff. This percentage was then applied to the natural channel to be cut off and the prototype-to-model stage relation curve for this location was applied to the location of the headwater control (Appendix B).

Sediment input

29. The influence of the proposed lock and dam on stages will extend beyond the upstream limit of the model. Thus, because stages upstream of the model limits will be influenced, the rate of sediment transported into the model reach prior to construction of the lock and dam will not be the same as that immediately after construction of the lock and dam. Over time, adjustments to the prototype channel geometry beyond the limits of the model will occur, and the rate of sediment transport will be adjusted to approximately that of preproject conditions. Therefore, the sediment input curve developed during adjustment to natural conditions did not apply to all anticipated postproject conditions. To model the anticipated postproject conditions, sediment was input based on the model operator's judgment for the first reproduction of the typical hydrograph, which is the same procedure used in the adjustment process. Based on the difference between the amount of sediment input during the first hydrograph and that input for natural conditions, additional sediment was added in even increments during each of the next four hydrographs. Thus, by the fifth hydrograph the amount of sediment being input into the model was based on preproject conditions. This is not to say that all of the channel adjustments upstream will occur in 5 years, as no simulation techniques were used to determine how long these adjustments will require. However, the described technique was believed to be a better approximation than adding the amount of sediment based on preproject conditions beginning with the first postproject hydrograph.

Supplemental slope

30. During the adjustment process, supplemental slope is added to a model to achieve the desired sediment movement. Supplemental slope is slope added to the model beyond the additional slope resulting from the distorted geometric scales. The supplemental slope added to the model is normally fixed based on the final adjustment run. However, the supplemental slope for the cutoff channels, not present during the adjustment process, had to be

determined based on the supplemental slope of the natural channel. Two alternatives, (a) maintaining the same supplemental slope in the cutoffs as in the natural channel and (b) maintaining the same total drop in elevation through the cutoffs as in the natural channel, were analyzed.

31. After careful examination of the effect of the supplemental slope in a cutoff, a practical qualitative solution was reached. By using the same supplemental slope in the cutoffs as in the natural channel, the effect of the supplemental slope on sediment transport would remain constant. Implementation of this alternative presented no serious problems for the present study, because both natural channels were being closed. However, if a natural channel were to stay open, the distance around the natural channel would be much greater than the distance through the cutoff. If the supplemental slope were kept constant in both the cutoff and natural channel, a discontinuity would occur where the cutoff rejoined the natural channel. This discontinuity is unacceptable. Changing the supplemental slope in the natural channel could not be justified as it had been adjusted to reproduce prototype behavior, nor could it be justified to model the present situation differently from one in which the natural channel was not closed off. Therefore, it appeared that the proper alternative was to keep the total drop in elevation constant through the cutoffs.

32 Keeping the total drop in elevation constant is not only sound from a practical modeling point of view but can be verified using river mechanics based relationships. Models are adjusted to reproduce prototype behavior, and thus the controlling parameters in the model must be proportional to those in the prototype. Using three different parameters important to sediment transport and river morphology, it can be shown that the total drop in elevation should be kept constant.

33. By adjusting the model to reproduce the bed configurations of the prototype, it can be assumed that the stream power in the model is proportional to the stream power in the prototype for a given discharge in order to obtain similar bed configurations (Richards 1982). Stream power is defined to be γRSV where γ is the specific weight of water, R is the hydraulic radius, V is the velocity, and S is the slope. By keeping all variables constant except for slope and using Manning's equation, V is shown to be proportional to $CS^{0.5}$ where C is a constant. This assumption simplifies

stream power to be $CS^{1.5}$. If the stream power in the model is proportional to the stream power in the prototype, then

$$\text{constant} = \frac{(S^{1.5})_p}{(S^{1.5})_m} \tag{2}$$

and it can easily be seen that if the slope in the prototype is increased by some factor, the slope in the model must also be increased by the same factor. Therefore, if the slope approximates the bed slope, this reasoning supports the previous conclusion that the total drop in elevation must be maintained in the model, because the total drop in elevation is maintained in the prototype.

34. The same result can be obtained by looking at Lane's relationship (Lane 1955): $Q_s d \propto Q_w S$, where Q_s is sediment discharge, d is median sediment size, Q_w is discharge, and S is slope. Assuming that the sediment discharge in the model should be proportional to the sediment discharge in the prototype yields

$$\frac{(Q_s)_p}{(Q_s)_m} \propto \frac{(Q_w S d^{-1})_p}{(Q_w S d^{-1})_m} \tag{3}$$

By holding all variables constant except for slope, the prototype and model slope must both change by the same factor. Similar results can also be obtained by assuming that the Shields parameter F_s in the prototype is proportional to the Shields parameter in the model, and all variables except slope are constant and represented by C :

$$\frac{(F_s)_p}{(F_s)_m} = \frac{(CS)_p}{(CS)_m} \tag{4}$$

35. Based on stream power, Lane's relationship, and the Shields parameter, it has been shown that the total drop in elevation in the model prior to the cutoff should be maintained after the cutoff. Therefore, the previous qualitative solution is supported by basic parameters important to river mechanics and sediment transport.

General Plan Description

36. The purpose of the plan described herein is to develop an

acceptable navigation channel approximately 9 ft deep and 200 ft wide. The normal pool will be at el 120 and the minimum tailwater at el 95 (the pool elevation of Lock and Dam No. 3). Therefore, to maintain acceptable depth for navigation, the maximum elevations of the bed upstream and downstream of the dam are approximately el 110 and el 85, respectively.

37. The natural channel upstream and downstream of the cutoff containing the lock and dam structures was realigned by the Vicksburg District to facilitate better navigation conditions. The alignment is shown in Plate 14 and described in detail in Table 1. The curves described in Table 1 are referenced to the Structure Azimuth Line (SAL). The SAL is defined to be at an elevation 3 ft above the Annual Low Water Plane (ALWP). For the purposes of modeling this reach of river, the ALWP was assumed to be constant at el 95 upstream of the dam and el 93 downstream of the dam. Therefore the SAL in the pool is el 98 and the SAL downstream of the dam is el 96. The revetted banks above the SAL were graded to a 1V on 4H slope while the revetted banks below the SAL were graded to a 1V on 1.5H slope. The Bull, Piermont, and Crain revetments were not changed during the testing program, although transverse and curved longitudinal dikes were incorporated into the final plan.

38. The proposed structures consist of a lock with upstream and downstream guard walls, a tainter-gated spillway, and a hinged-crest-gated spillway (Plate 15). The lock consists of a lock chamber 84 ft wide by 685 ft of usable length. The upstream guard wall is a ported riverside wall beginning at sta 1+75.0 and extending 700 ft upstream to sta 8+75.0. The downstream guard wall is a solid riverside wall beginning at sta 9+19.5 and extending 650 ft downstream to sta 15+69.5. The tainter-gated spillway is separated from the lock by a distance of 69 ft and consists of four tainter gates, 60 ft wide by 36 ft high, with a sill elevation of 85.0. The hinged-crest-gated spillway is located adjacent to the tainter-gated spillway and consists of three 100-ft hinged-crest gates with a crest elevation between el 115 and el 122. The structures are located such that the azimuth of the center line of the lock is 301°48'47" and the intersection of this center line with the axis of the dam is located at N463,828.91, E1,761,006.79. All side slopes in the upstream excavated approach channel (hereafter referred to as the approach channel) and the downstream excavated exit channel (hereafter referred to as the exit channel) were protected against erosion. The basic configuration of approach channel remained unchanged throughout the testing program. However,

Table 1

Alignment Curves

Curve Name	Radius, ft	Point of Curvature	Point of Tangency	Center
Bull Revetment #2	8,000.0	Outside Model Limits	N471,470.0 E1,750,020.0	N469,706.1 E1,742,216.9
Bull Revetment #3	3,000.0	N471,470.0 E1,750,020.0	N470,398.5 E1,750,025.5	N470,919.0 E1,747,071.0
Piermont Revetment #1 (outside)	3,150.0	N469,380.7 E1,748,957.1	N465,842.9 E1,753,459.1	N468,645.0 E1,752,020.0
Piermont Revetment #1 (inside - not revetted)	2,550.0	N467,909.5 E1,749,578.4	N466,377.4 E1,753,186.4	N468,645.0 E1,752,020.0
Piermont Revetment #2	4,348.9	N466,377.4 E1,753,186.4	N466,221.6 E1,757,419.1	N462,503.7 E1,755,163.0
Crain Revetment #1	12,022.5	N462,110.0 E1,763,793.2	N459,044.9 E1,766,435.9	N452,804.5 E1,756,180.8
Crain Revetment #2	11,022.5	N459,044.9 E1,766,435.9	Outside Model Limits	N453,311.9 E1,757,046.4

the configuration of the exit channel required significant change to reduce velocities and scour.

Plan A

Description

39. Plan A (Plate 16) was the first plan tested in the movable-bed physical model. This plan consisted of a kicker dike on the left bank between ranges 6 and 10 and a berm in the upstream approach to the lock. The kicker dike consisted of an extension of the 3,000-ft-radius Bull Revetment No. 2 to N469,630.0, E1,749,780.0, then a 950-ft straight section extending tangentially from the curve. The downstream end of the natural channel near range 25 was closed off by a longitudinal dike connected to a transverse dike. The longitudinal dike was 280 ft long and tangent to Piermont Revetment No. 1.

The transverse dike tied to the longitudinal dike on the riverward end and the levee on the landward end. All dikes were level crested at el 125. In the approach channel a berm 100 ft wide at el 100 was left unexcavated. This berm tied to the natural channel on the upstream side with a 1V on 4H side slope with the upstream end configured parallel to the excavated bank line as shown in Plate 16. The right side of the berm was sloped at 1V on 4H to el 90, which is the bed elevation of the approach channel. On the downstream end of the berm, a 1V on 25H slope was used to transition the el 100 berm to the el 90 bed inside the guard wall. The entire excavated upstream approach channel and all but the most downstream 150 ft of the excavated downstream exit channel were protected against scour.

Results

40. Due to model operation problems during the plan testing, no data are presented for Plan A. However, qualitative observations of the tests showed excessive scour in the unprotected portion of the exit channel.

Plan B

Description

41. Plan B, shown in Plate 16, is identical in alignment to Plan A, but has changes in the extent of bed protection in the approach and exit channels. The bed protection in the approach channel ended about 40 ft upstream of the bull nose of the guard wall. The unexcavated berm along the left bank was left unprotected and therefore free to scour. No attempt was made to simulate the prototype erosion resistance of this unexcavated berm; it was modeled the same as any other movable-bed portion of the model. The bed protection in the exit channel was extended along the entire length of the excavated channel. The entire model was remolded to the initial conditions shown in Plate 11 before initiation of the tests.

Results

42. The test results of Plan B after each of the five repetitions of the typical hydrograph (Plate 10) are shown in Plates 17-21. Beginning at the upstream end of the model and working downstream, the following tendencies were observed.

<u>a</u>. A bar on the right bank between ranges 3 and 7 gradually increased in elevation throughout the tests and after the fifth

repetition had increased more than 10 ft to el 112. The base test also displayed this tendency, which suggests that the bar is not a result of realigning the natural channel.

b. The cutoff channel between ranges 10 and 25 formed a typical bend cross section, and resultant flow pattern, by scouring the bed at the outside of the bend and depositing a point bar on the inside of the bend. The point bar did not develop to an elevation that would prevent navigation over it, except between ranges 20 and 25. However, between ranges 20 and 25, the point bar gradually developed such that by the fourth run the navigation channel had been reduced from approximately 600 ft to 400 ft and by the fifth run to approximately 350 ft.

c. Immediately downstream of range 25 the flow leaving the cutoff channel reentered the natural channel and experienced a sudden expansion. This sudden expansion in cross-sectional area resulted in multiple bars and channels in this region. Although navigation depth was always maintained, the bed configuration was very unstable.

d. The large point bar in the natural channel between ranges 27 and 35 steadily increased in elevation with each repetition of the hydrograph. During high discharge (90,000 cfs and above), flow passed easily over this point bar; however, at low discharge the flow divided, with the majority of the flow remaining in the main channel to the left of the bar and a portion of the flow passing to the levee side of the bar, reentering the main flow at the upstream end of the lower cutoff. During high discharge the flow over the point bar funneled into the approach channel at the upstream end of the approach channel causing significant crossflow. This crossflow from the right portion of the channel struck the point on the right bank at the upstream end of the approach channel and caused flow separation and the formation of an eddy. The eddy caused scour in the immediate vicinity of the point and deposition of this scoured material further downstream along the right bank of the approach channel.

e. The unexcavated and unprotected berm along the left bank of the approach channel was completely removed by scour during the first hydrograph. Initially the downstream end of the berm began to move and deposit inside the upstream guard wall; however, the first 125,000-cfs flow created strong enough velocities to transport the sediment out from behind the guard wall through the ports. No further deposition problems behind the guard wall were observed for the remainder of the hydrographs.

f. During the first hydrograph, the sediment load in the vicinity of the dam was very high due to the erosion of the berm in the approach channel. However, after the first hydrograph, the sediment load in the vicinity of the lock and dam was greatly reduced. The reduced sediment transport into the vicinity of the dam caused some scour of the approach channel and significant scour in the natural channel just downstream of the end of the bed protection in the exit channel. The area downstream of

the exit channel (ranges 55 to 57) scoured as deeply as the model would permit during the second and third hydrographs. During the fourth hydrograph, sediment from the upstream natural channel began moving into the approach channel and through the dam. The increase in sediment load caused some aggradation of the approach channel but did not adversely affect the navigation channel. Although the increased sediment load through the dam resulted in some shoaling of the scour hole downstream of the exit channel, the hole remained about 30 ft deeper than the bed protection in the exit channel.

g. The last few ranges of the model survey show that deposition occurred that eliminated sufficient navigation depth. However, this portion of the model, due to physical constraints around the model, did not properly reproduce the Crain Revetment alignment; therefore, the results in this area may not be reliable.

Plan C

Description

43. Plan C (Plate 22) contains several changes to the configuration described in Plan B. The kicker dike in Plan B between ranges 6 and 10 was replaced by a series of four spur dikes (Table 2) to facilitate requirements for navigation. The longitudinal section of the closure dike in Plan B, near range 25, was modified to eliminate the straight segment and extended to form a curved longitudinal dike. The curved longitudinal dike tied to Piermont Revetment No. 1 at the point of tangency, with the same center as Piermont Revetment No. 2, a radius of 3,148.9 ft, and a point of tangency at N466,202.65, E1,754,553.33. In addition, two transverse dikes were added between ranges 30 and 35 (Table 2) to avoid the sudden expansion, thus maintaining a more uniform channel width.

44. The downstream exit channel was widened 50 ft to a bottom width of 350 ft to reduce high velocities that were adversely impacting navigation. This wider channel was constructed by keeping the left bank the same and moving only the right bank to provide a smoother alignment than that used in Plans A and B. The right bank realignment began at the dam with a line parallel to the lock from the stilling basin up the sloped channel to el 81. The right bank was then angled so that it was tangent to a 7,015-ft-radius curve, having a center at N456,382.7, E1,759,127.1 and a point of curvature at N462,592, E1,762,391. This curved right bank was maintained until the exit

Table 2

Plan C Dike Descriptions

Dike Description	Riverward End	Azimuth from Bank
Bull Dike No. 1	N469,876	159°47'22"
	E1,749,941	
Bull Dike No. 2	N469,396	159°47'22"
	E1,749,803	
Bull Dike No. 3	N468,915	159°47'22"
	E1,749,665	
Bull Dike No. 4	N468,458	159°47'22"
	E1,749,590	
Piermont Dike No. 1	N466,059	281°36'38"
	E1,755,609	
Piermont Dike No. 2	N465,899	292°57'21"
	E1,756,585	

channel intersected the natural channel. The left edge of the berms at el 90 and el 114 were adjusted to account for the change in alignment while maintaining the 1V on 4H side slope. All side slopes were protected against erosion throughout the exit channel, but the bed protection ended 300 ft downstream of the downstream end of the guard wall.

45. The entire model was remolded to the initial conditions (Plate 11) prior to initiation of testing.

Results

46. Results of testing Plan C for runs 1, 5, and 10 are shown in Plates 23-25, respectively. Similar to the results of Plan B, a bar formed between ranges 3 and 7 to an elevation greater than el 110 in 5 runs and after 10 runs had reached el 114. The cutoff channel between ranges 10 and 25 again formed a typical bend cross section by scouring the bed at the outside of the bend and depositing a point bar on the inside of the bend. The only encroachment on navigation upstream of the dam occurred between ranges 10 and 15 immediately downstream of the last spur dike, where a bar formed to el 113. The bar that formed near range 25 in Plan B did not form at any time during the testing of Plan C. The curved longitudinal dike and two transverse dikes opposite Piermont Revetment No. 2 were successful in providing a consistent top width,

which resulted in a deeper and wider navigation channel through the bend. In addition, these dikes eliminated the strong crossing flow at the right bank of the upstream end of the approach channel that occurred in Plan B. Some filling along the right side of the approach channel was observed between ranges 40 and 45, and some scour occurred along the left bank between ranges 35 and 40. In the exit channel excessive scour was again observed beyond the downstream end of the bed protection.

Plan D

Description

47. No changes were made to the upstream portion of the model. The exit channel was modified by removing the existing el 90 and el 114 berms and adding a 200-ft-wide berm at el 100 to the right bank (Plate 26). The el 100 berm was constructed parallel to the right bank described in Plan C. The side slopes on the berm were 1V on 4H. The upstream face of the berm was sloped at 1V on 25H from the downstream end of the hinged-crest stilling basin until el 100 was reached. The downstream face of the berm intersected the natural channel and maintained the existing slope of the natural bank. All side slopes and the entire el 100 berm were protected against erosion. The bed protection was stopped 300 ft downstream of the downstream end of the guard wall as in Plan C.

48. The upstream portion of the model was not remolded for the remainder of the testing program. Therefore, Plan D, run 1, is actually run 11 for the upstream portion of the model. Table 3 shows the relationship of the plan descriptions and run numbers to the actual number of runs on the upstream portion of the model.

Results

49. The results of testing Plan D are shown in Plates 27-29. In the portion of the model upstream of the dam, the bar at range 25 that developed in Plan B began to enlarge and reduce the width of the navigation channel at this location. In the portion of the model downstream of the dam, an eddy formed off the end of the bed protection. The eddy caused a bar to build along the left bank just downstream from the end of the bed protection between ranges 51 and 52. This bar was the result of sediments being scoured by the eddy from the center portion of the channel and being deposited near the left bank. The maximum elevation of the bar was el 99, although by the end of the

Table 3

Conversion of Plan Description and Run Numbers for the Upstream Portion of the Model

Plan Description	Run Number	Upstream Run Number
Plan C	1	1
Plan C	5	5
Plan C	10	10
Plan D	1	11
Plan D	2	12
Plan D	3	13
Plan D-1	1	14
Plan D-1	2	15
Plan D-1	3	16
Plan D-2	1	17
Plan D-2	2	18
Plan D-3	1	19
Plan D-3	2	20
Plan D-3	3	21
Plan D-3	4	22
Plan D-3	5	23
Plan D-3	6	24
Plan D-3	7	25

hydrograph, the elevation of the bar was usually reduced to below el 85. However, it is anticipated that this bar could present navigation problems during certain flows in a hydrograph due to insufficient navigation depth.

Plan D-1

Description

50. Plan D-1, shown in Plate 26, is similar to Plan D, with the exception that the bed protection was extended 500 ft to 800 ft downstream of the downstream guard wall bull nose. Only the downstream portion of the model was remolded. However, the bar that developed along the left bank at the entrance

to the cutoff channel near 1967 river mile 212 (ranges 11 to 15) had shoaled to an elevation that presented a potential navigation problem. Therefore, prior to testing Plan D-1 this bar was dredged to el 105.

Results

51. The results of Plan D-1 are shown in Plates 30-32. The bar near 1967 river mile 212 re-formed during the first hydrograph. A rough simulation of this bar was constructed in the navigation model to determine its potential effects on navigation. Preliminary evaluation of the bar showed no adverse effects on navigation; therefore, no further efforts were made to eliminate the bar.

52. The eddy off the end of the bed protection in Plan D now formed off the end of the extended bed protection, but the resultant deposition had been reduced to an acceptable level. However, significant deposition was observed on the bed protection in the navigation channel between ranges 50 and 52 (Plates 31 and 32). This deposition occurred along a line from the bull nose to the intersection between the end of the bed protection and the left bank. This line of deposition was formed at the shear zone between the still water behind the guard wall and the flowing water in the channel. The deposition resulted in insufficient depth in the navigation channel.

Plan D-2

Description

53. Plan D-2 is shown in Plate 33. A notch was put in the closure dike behind the longitudinal dike near range 25 to provide access to the natural channel. This notch was 300 ft wide with a base at el 110. In the downstream portion of the model, the berm on the right bank was modified to include a berm at el 90 immediately downstream of the hinged-crest gates, as in Plan C. A 1V on 4H slope was used between the el 90 and el 100 berms. A wing dike was added at the lower end of the downstream guard wall. The dike was 225 ft long, level crested at el 97, and had an azimuth of 136°48'47" measured from the upstream end of the dike. To provide additional room for upbound tows approaching the lock, the left bank was realigned. The bank line landward of the guard wall was extended 650 ft downstream of the bull nose. A curve having the following characteristics was used to connect the end of the extension to the upstream end of the Crain Revetment:

Radius to SAL	8,160.0 ft
Point of Curvature	N462,772.8, E1,762,403.2
Point of Tangency	N462,275.7, E1,763,651.7
Center	N455,918.4, E1,758,536.0

Results

54. The results of testing Plan D-2 for two hydrographs are shown in Plates 34 and 35. In the upstream portion of the model a bar between ranges 23 and 25 began to form and showed tendencies of shoaling to an elevation that would impact navigation. In the downstream portion of the model, an eddy continued to form off the downstream end of the bed protection. The eddy scoured the bed from the center of the channel and caused a bar to be built along the left bank just downstream from the end of the bed protection. The maximum height of the bar was el 95; however, by the end of the hydrograph, the elevation of the bar was reduced to el 85 or below. Therefore, under certain hydrologic conditions a bar could form in the downstream approach that would require dredging to maintain adequate navigation depth.

Plan D-3

Description

55. Plan D-3, shown in Plate 33, is similar to Plan D-2 with the exception that the bed protection in the downstream approach was extended to the downstream end of the excavated channel and in the upstream portion of the model the 300-ft-wide notch for access to the old channel was moved from range 25 to the closure dike behind the reverse kicker near range 26.

Results

56. Through five repetitions of the typical hydrograph, Plan D-3 (Plates 36-40) showed no adverse sedimentation problems in the downstream portion of the model. The bar between ranges 23 and 25 that had begun developing in Plan D-2 was monitored throughout the testing of Plan D-3. After the fifth run on Plan D-3, the bar was dredged to el 105. Two runs were made to evaluate the tendency of the dredged bar to rebuild. The first run (Plate 41) showed a tendency to rebuild the dredged bar; however, the second run (Plate 42) removed the bar. Therefore, although this area has shown a tendency to build a bar and narrow the navigation channel, the problem does not seem to warrant a river training structure. Based on the model results the bar could be managed using maintenance dredging as necessary.

PART IV: DISCUSSION OF RESULTS AND CONCLUSIONS

Limitation of Model Results

57. In analysis and evaluation of the results of this study, the limitations of the model should be considered based on the model adjustment, base test, and hydrographs and water-surface elevations used. The adjustment of this model was based primarily on a single prototype survey (1978) which was preceded by abnormally low water. A hydrograph more typical of the normal range of flows was used in conjunction with this survey. The adjustment was verified, however, by running the extended low-water period following the 1978 prototype survey and ensuring that the model displayed tendencies similar to those shown in the 1980-81 prototype survey. Because of the significant changes resulting from the proposed improvement plan, the model base test in the preproject configuration was only of marginal use when interpreting the results of the proposed plans. The magnitude of the cutoffs and changes to the model from the preproject condition required extension of current modeling procedures that have not been verified by prototype data. The model does not reproduce the movement of material transported primarily in suspension, which is significant in the Red River, nor was any attempt made to reproduce the active bank erosion observed in the prototype. The typical hydrograph used accurately reflects the historical flow duration data, but natural hydrographs that deviate from the typical representation, when used in testing, may produce different bed configurations. No prototype stage or discharge information was available within the model reach; therefore, all stage and discharge data were interpolated from gage locations outside the modeled reach. The postproject tailwater elevations were predicted by the Vicksburg District using numerical simulations based on the proposed channel modifications. In spite of the limitations presented, the model adjustment and modeling techniques are considered sufficient to indicate trends that can be expected under the conditions imposed on each plan tested.

Summary of Results and Conclusions

58. The following results and general indications were developed from the model study:

a. Replacement of the kicker dike at the end of the Bull Revetment with four spur dikes had no adverse effect on the bed configuration.

b. The cross section through the upstream cutoff formed a typical bend cross section with significantly deeper water along the outside of the channel and a point bar along the inside. The point bar did not reach an elevation at which navigation depth was eliminated. However, the shape of the bar and cross section significantly changed the flow patterns in this bend from those of the trapezoidal section.

c. A curved longitudinal dike and two transverse dikes opposite Piermont Revetment No. 2 are necessary to maintain a uniform width channel, thus avoiding excessive expansion of the flow area and resultant deposition in the channel. These dikes also eliminated the eddy created by the sudden contraction as the flow entered the excavated lock approach.

d. Some maintenance dredging may be necessary to maintain a full-width navigation channel at the exit of the upstream cutoff channel near the transition from Piermont Revetment No. 1 to No. 2.

e. The berm along the left bank in the approach channel will erode to el 80 or below if the material is not resistant to erosion.

f. The excavated portion of the exit channel must be completely protected against bed scour to avoid the creation of a large scour hole and deposition of the scoured material along the left bank to an elevation that limits navigation.

g. When the excavated portion of the exit channel was 350 ft wide at el 81 with a 1V on 4H slope up to a 200-ft-wide berm at el 100 along the right bank, acceptable results were achieved. Without this widening, the velocities were very high and resulted in excessive scour and adverse navigation conditions.

h. A submerged wing dike placed off the end of the bull nose on the downstream guard wall prevented bed material from depositing in the lock approach.

i. There may be insufficient depth to permit navigation in the natural channel downstream of the lock and dam. Due to physical constraints in the model facility, this area was not exactly reproduced in the model. This area was also at the end of the model and therefore was subject to the effects of the exit conditions of the model. Acknowledging the inaccuracies of modeling this area, all tests showed a tendency for this area to lack sufficient depth for navigation.

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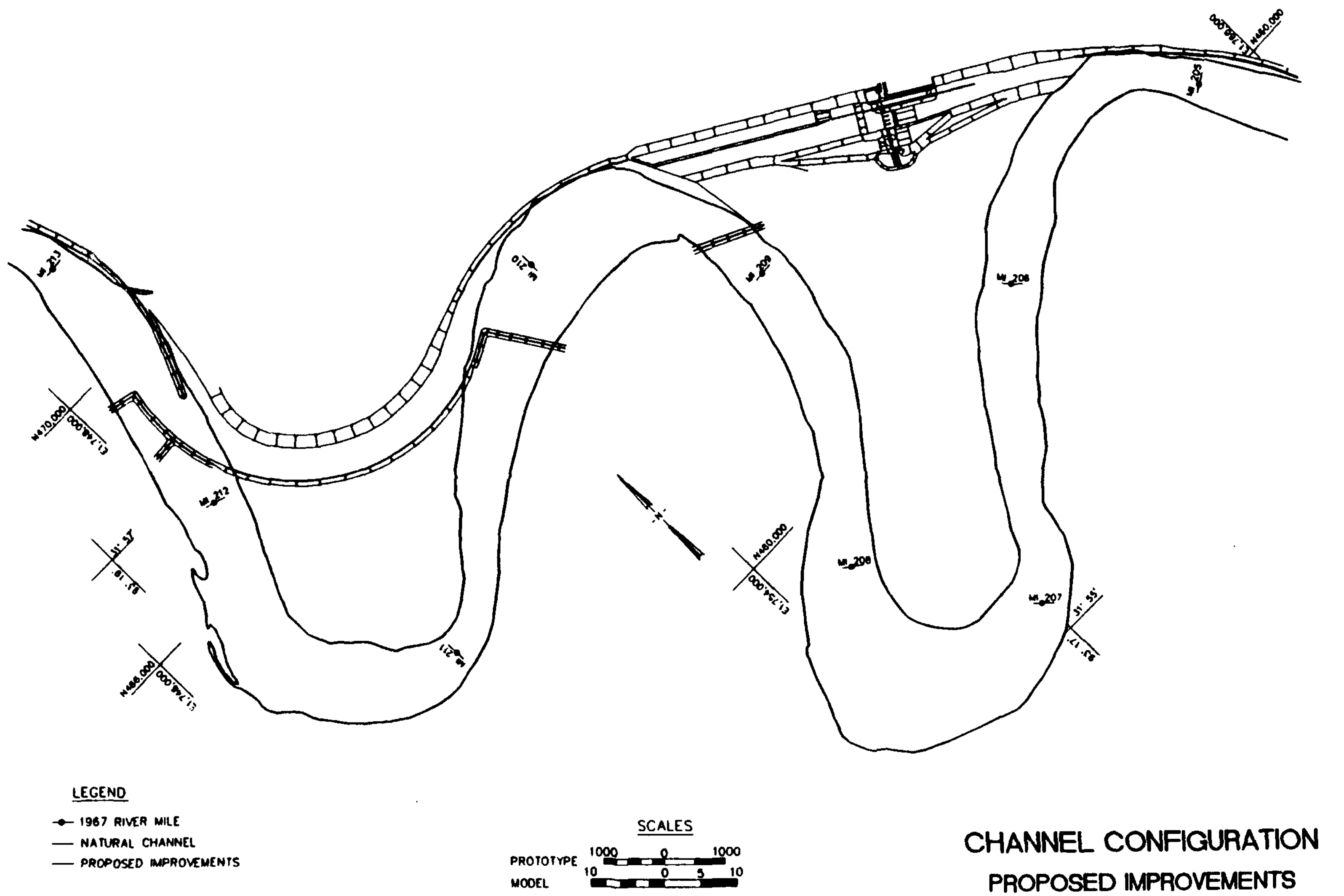
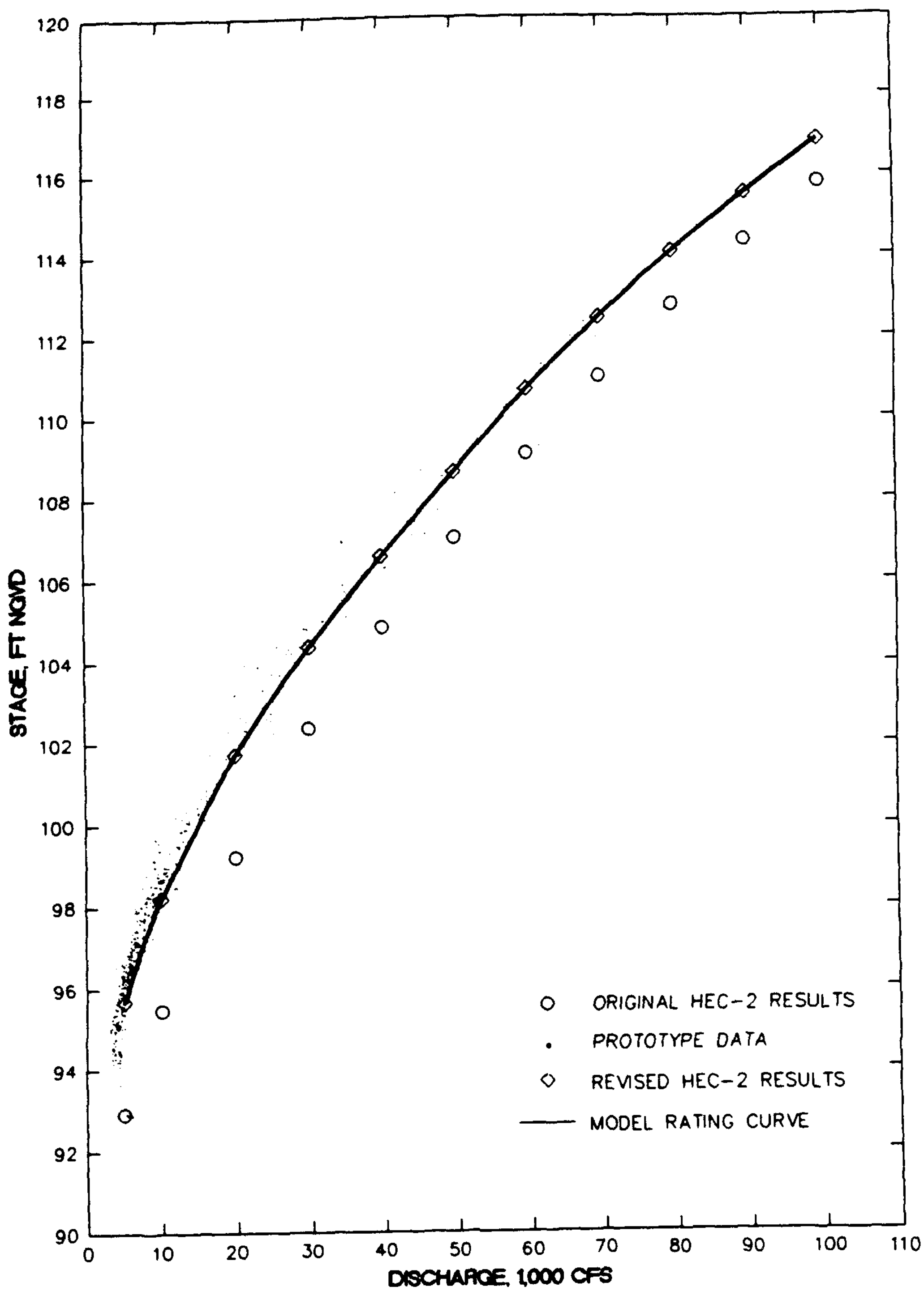


CHANNEL CONFIGURATION

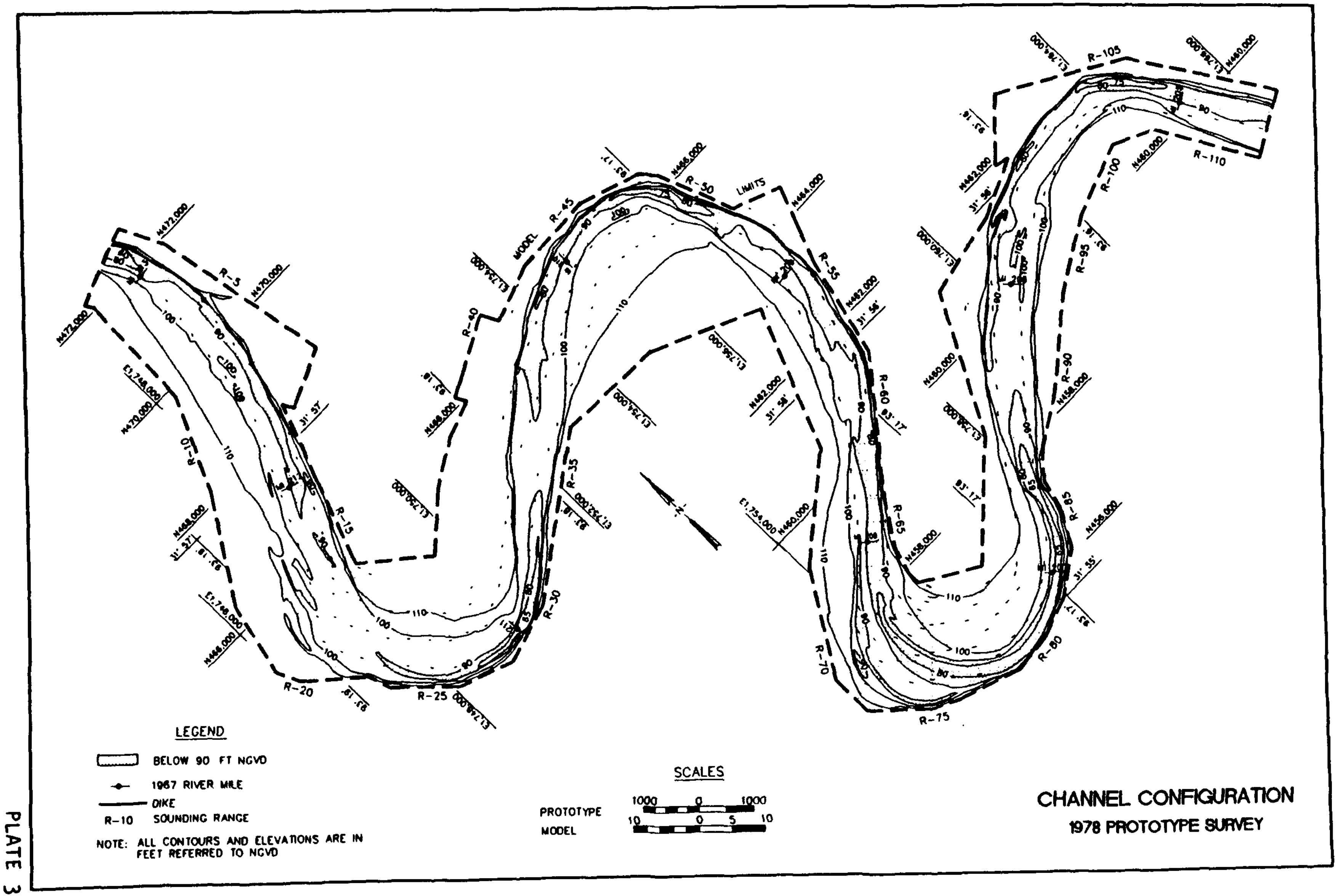
PROPOSED IMPROVEMENTS

PLATE 1



PREPROJECT RATING CURVE

PLATE 2



CHANNEL CONFIGURATION
1978 PROTOTYPE SURVEY

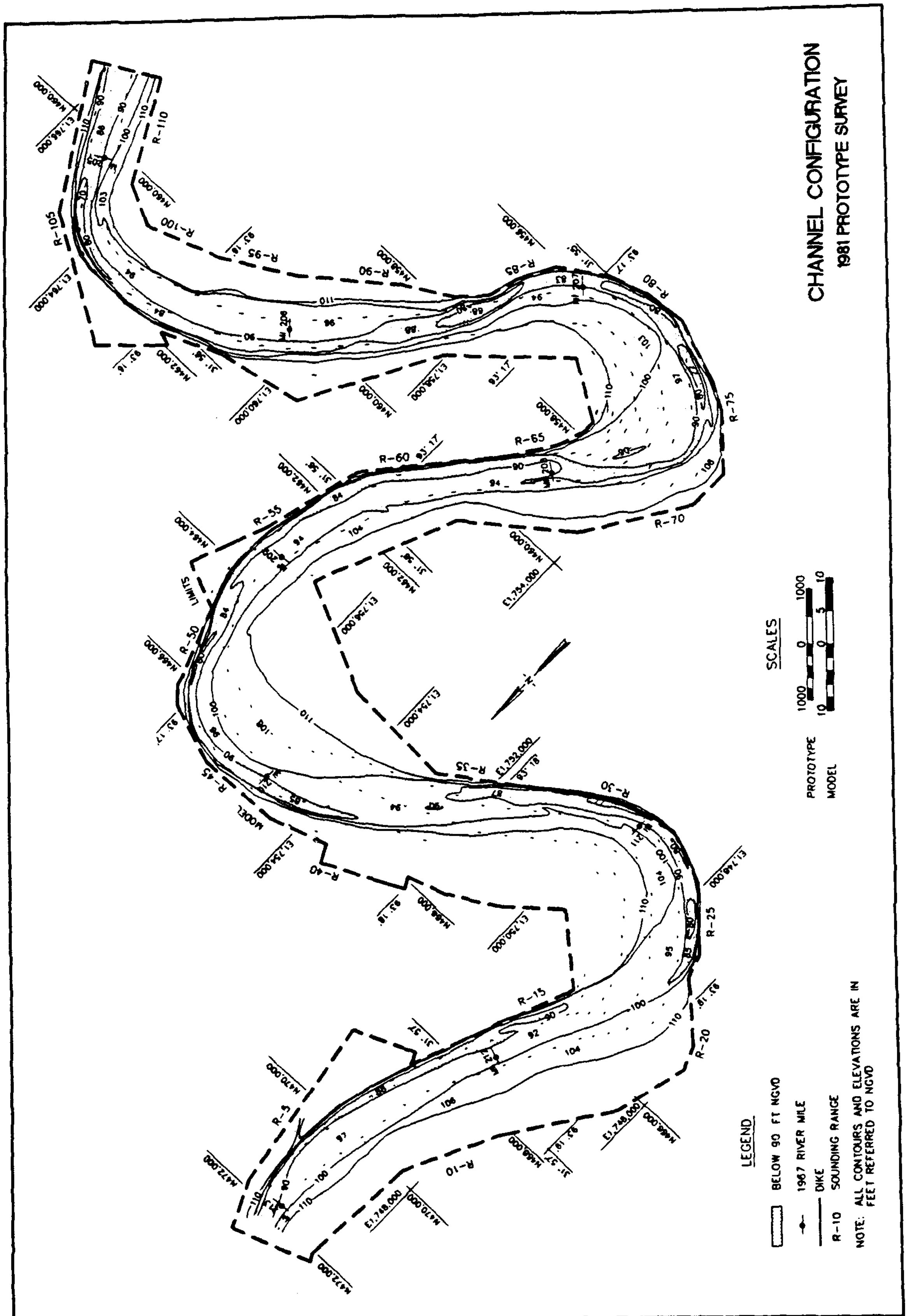
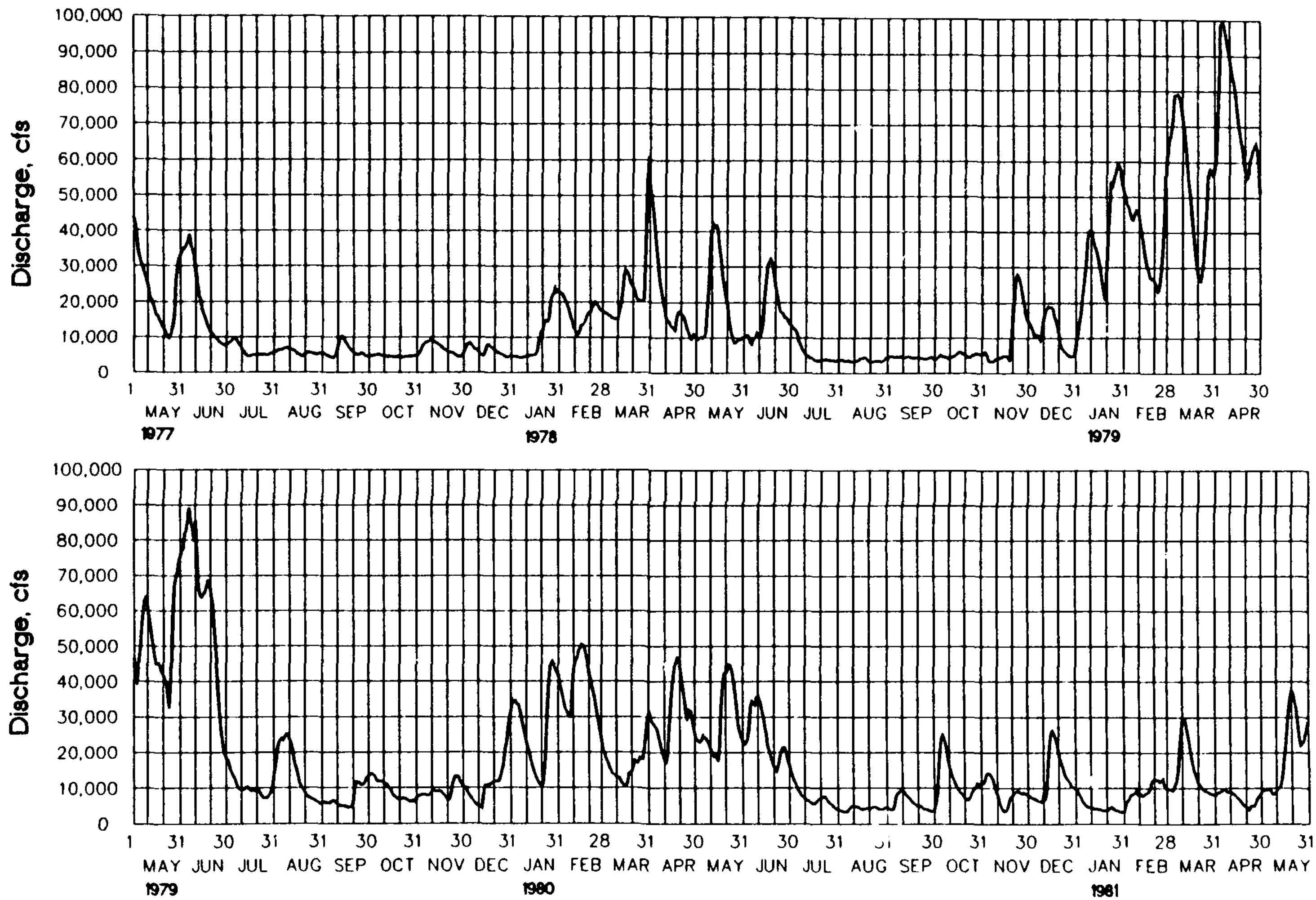
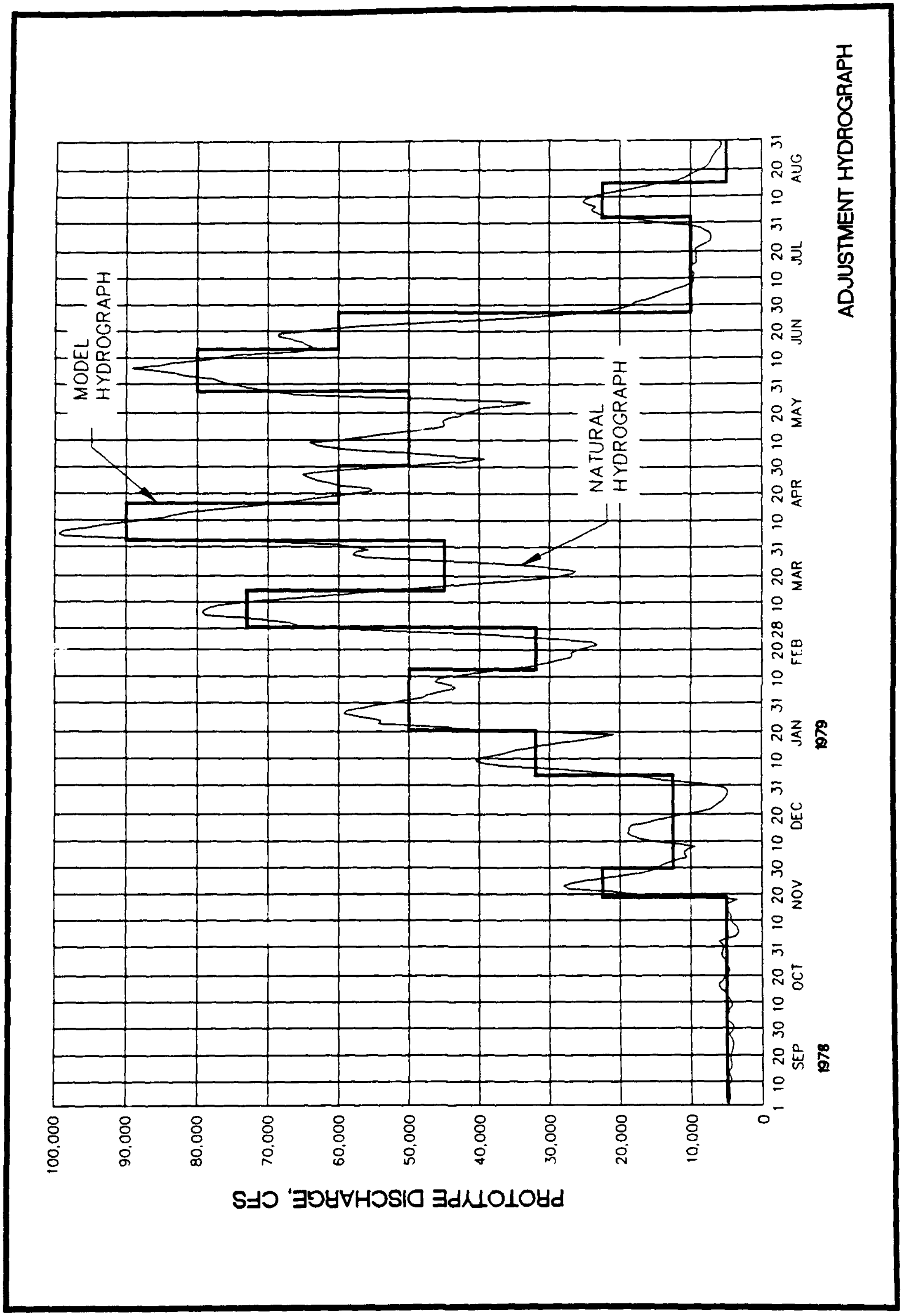

CHANNEL CONFIGURATION
1981 PROTOTYPE SURVEY
LEGEND
BELOW 90 FT NGVD
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 1000
MODEL 10 0 5 10
MODEL
LIMITS

PLATE 4



DISCHARGE AT 1967 RIVER MILE 209



ADJUSTMENT HYDROGRAPH

PLATE 6

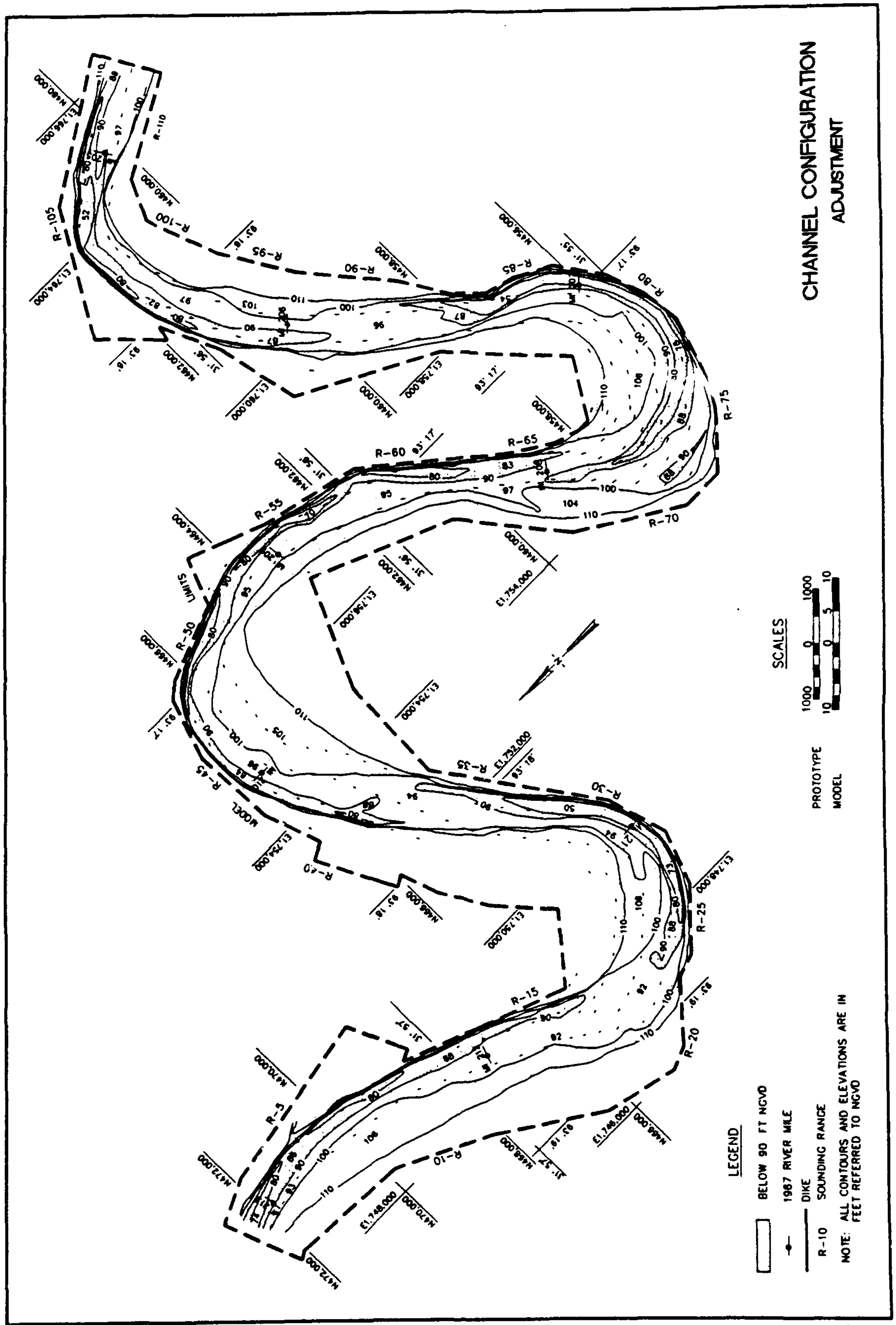
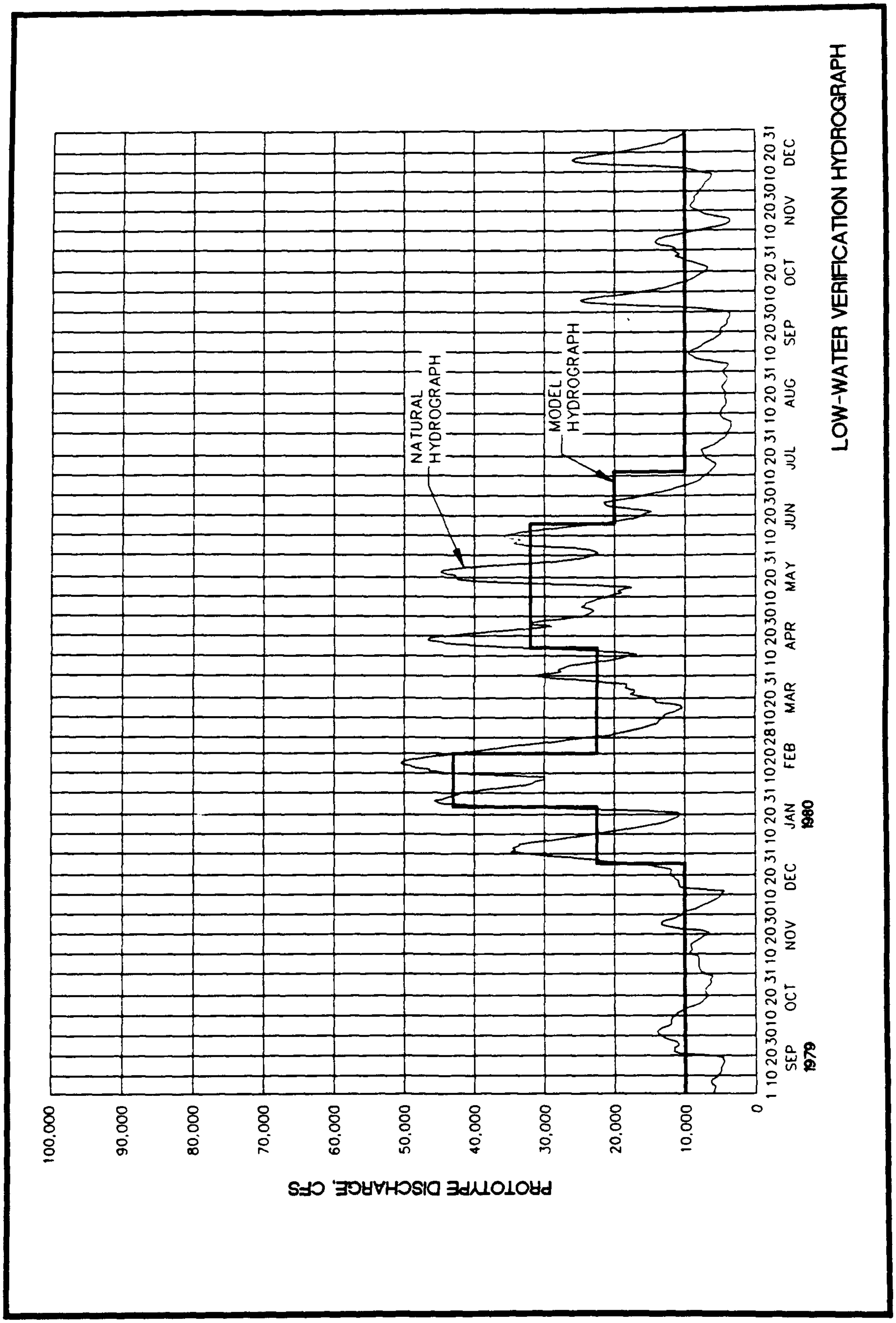
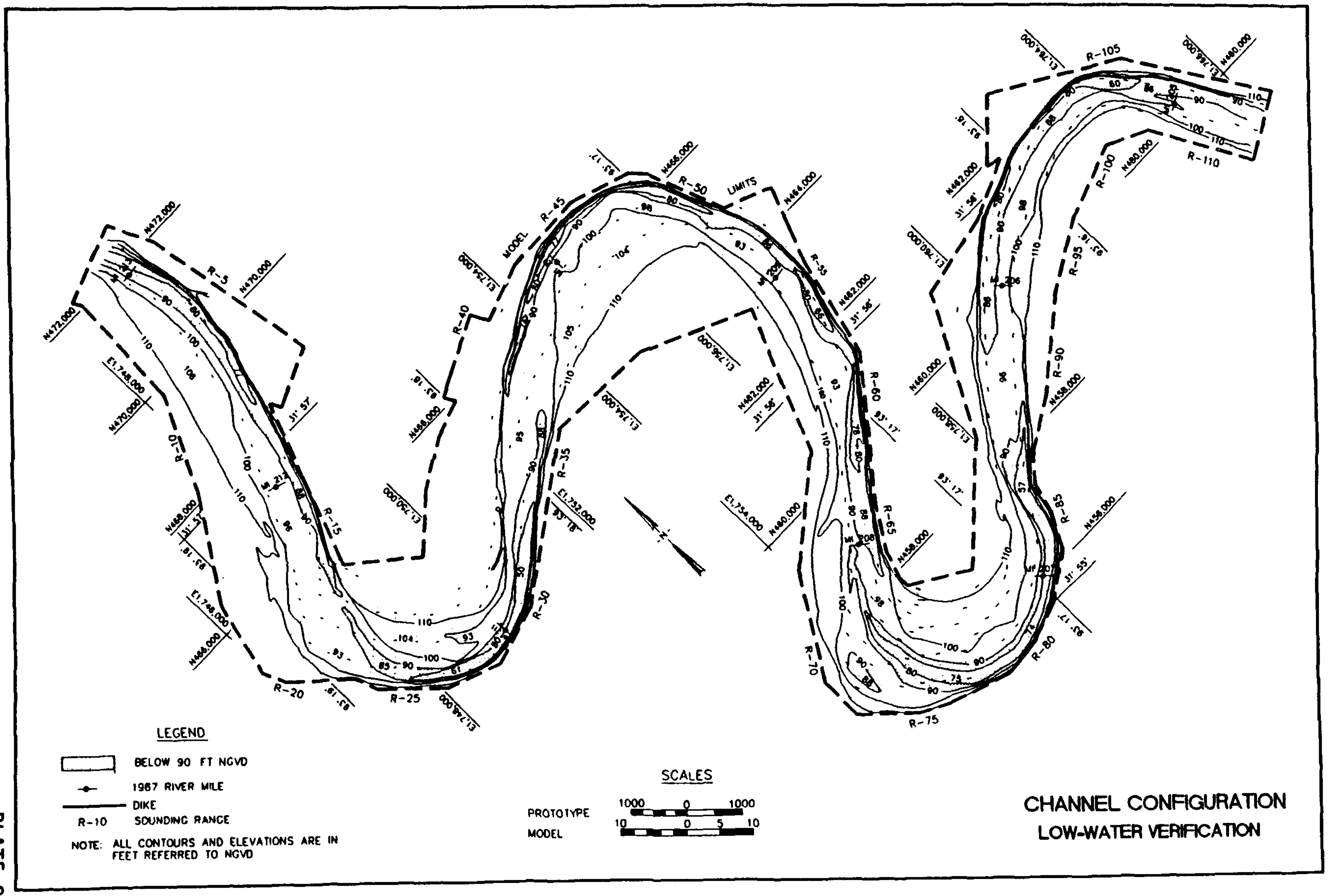

CHANNEL CONFIGURATION
ADJUSTMENT
LEGEND
BELOW 90 FT NGVD
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 1000
MODEL 10 0 5 10
MODEL LIMITS

PLATE 7



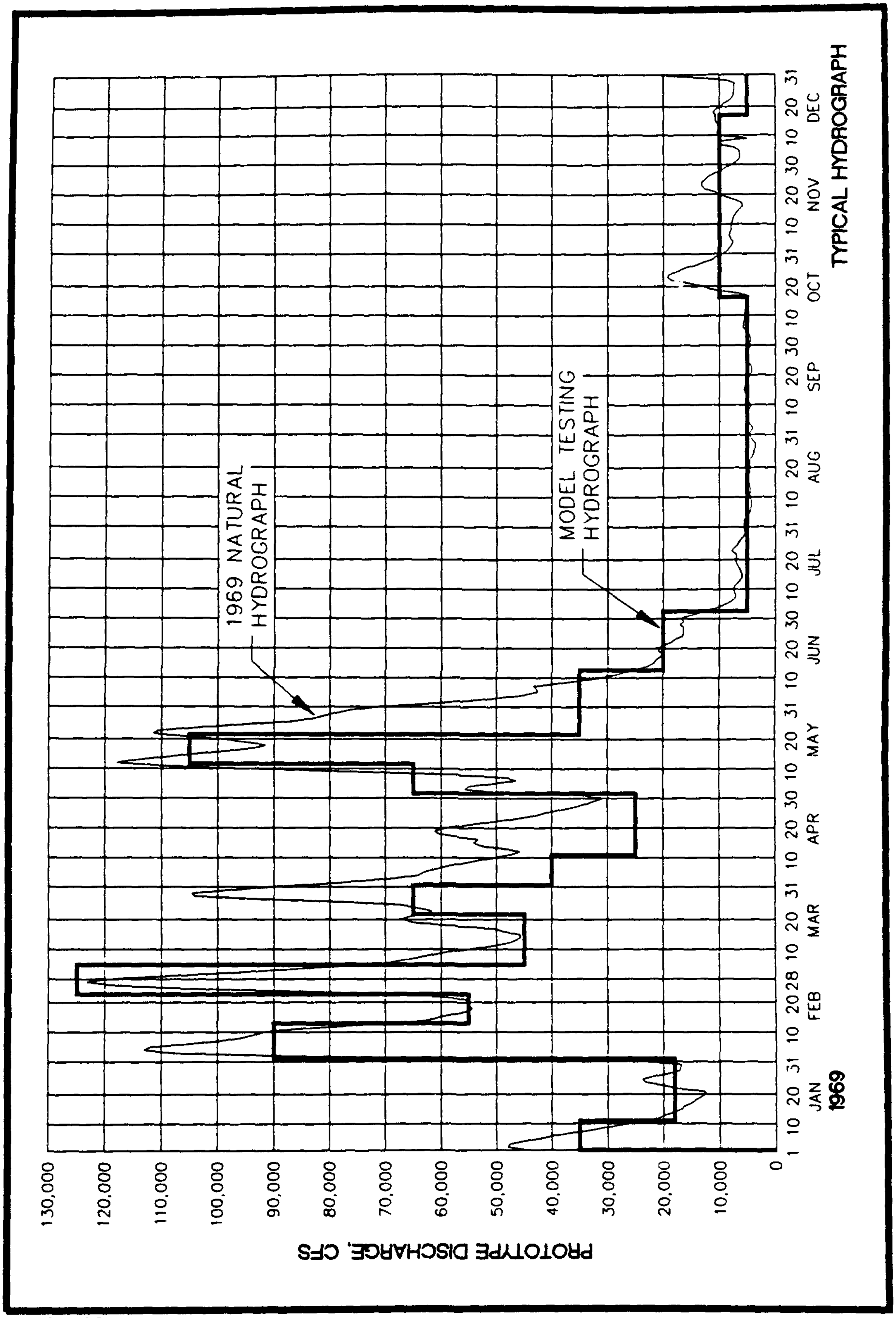
LOW-WATER VERIFICATION HYDROGRAPH

PLATE 8



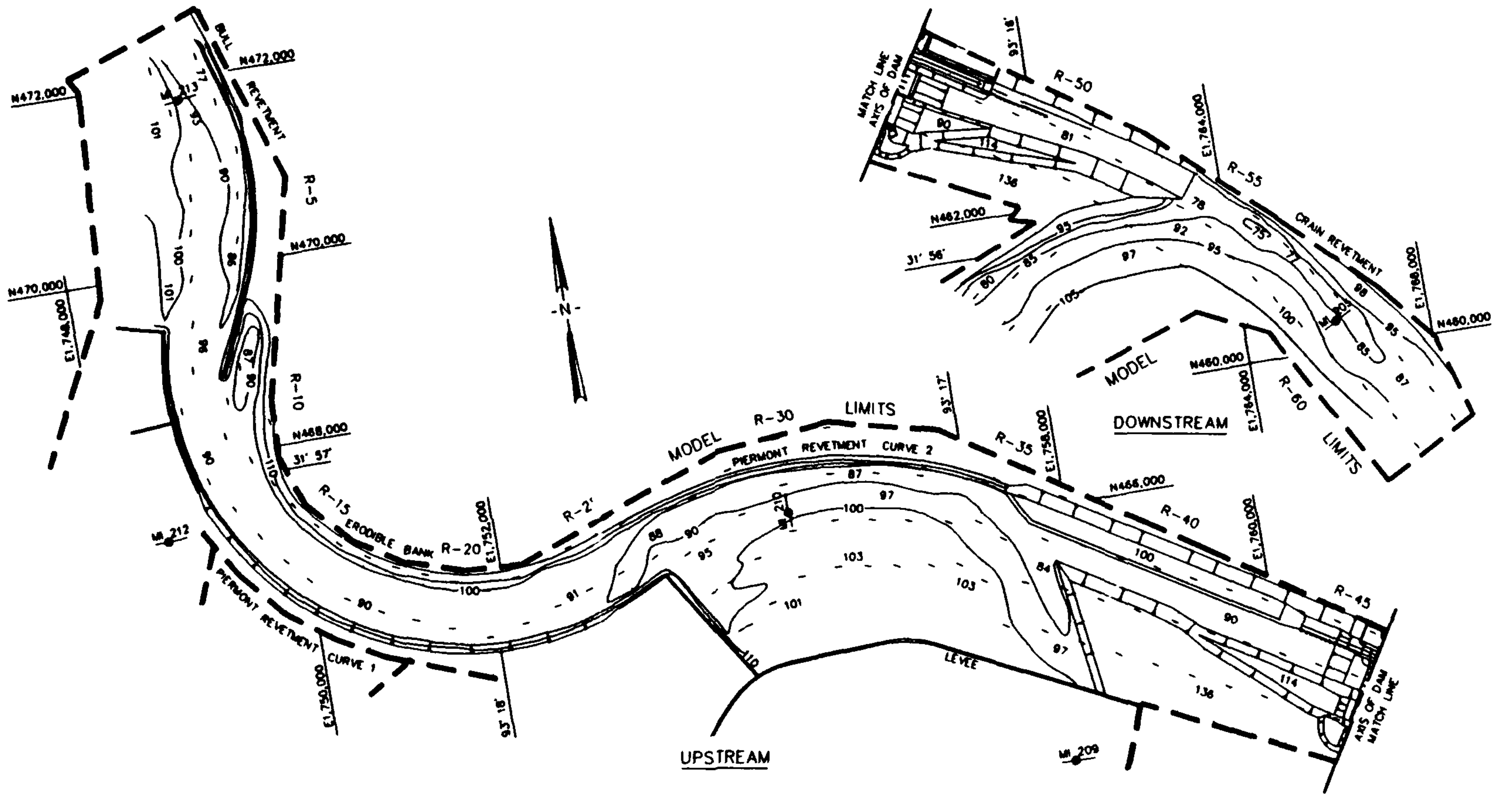
CHANNEL CONFIGURATION
LOW-WATER VERIFICATION

PLATE 9



TYPICAL HYDROGRAPH
1969 NATURAL HYDROGRAPH
MODEL TESTING HYDROGRAPH
PROTOTYPE DISCHARGE, CFS
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
1 10 20 31 10 20 28 10 20 31 10 20 30 10 20 31 10 20 30 10 20 31 10 20 31 10 20 30 10 20 31 10 20 30 10 20 31
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC
1969

PLATE 10



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

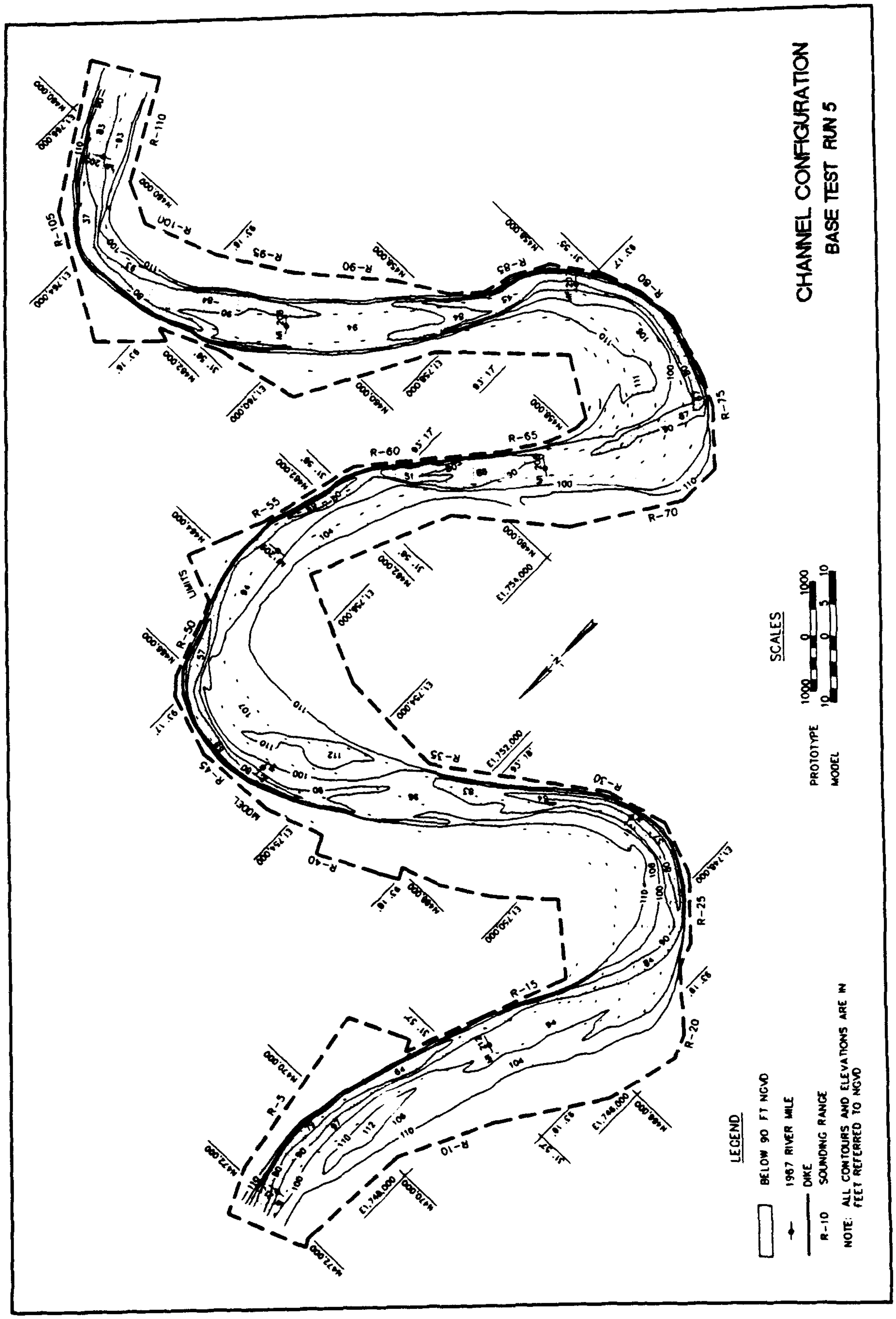
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

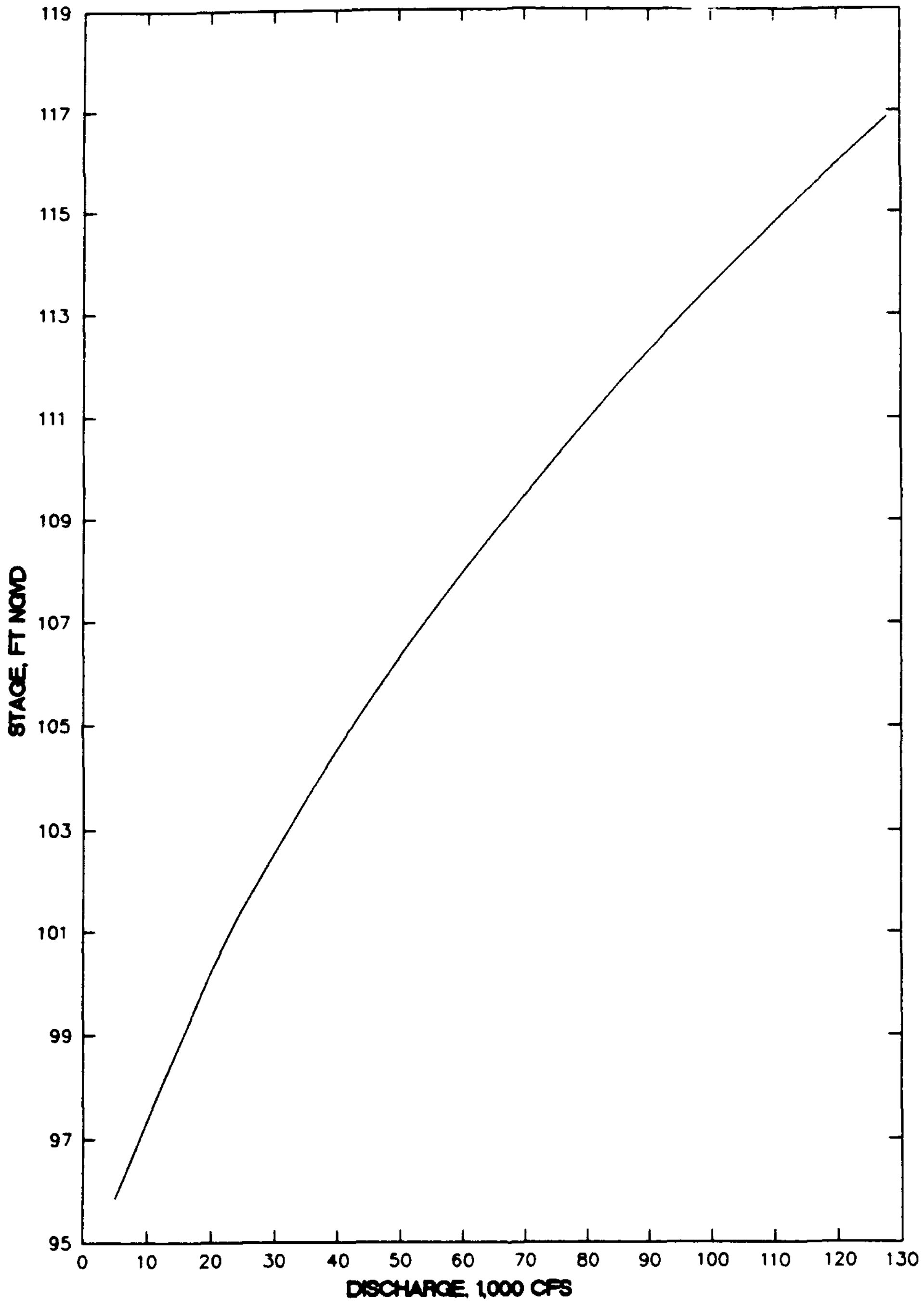
CHANNEL CONFIGURATION

POSTPROJECT INITIAL CONDITIONS



CHANNEL CONFIGURATION
BASE TEST RUN 5

PLATE 12



POSTPROJECT RATING CURVE
1967 RIVER MILE 204.8

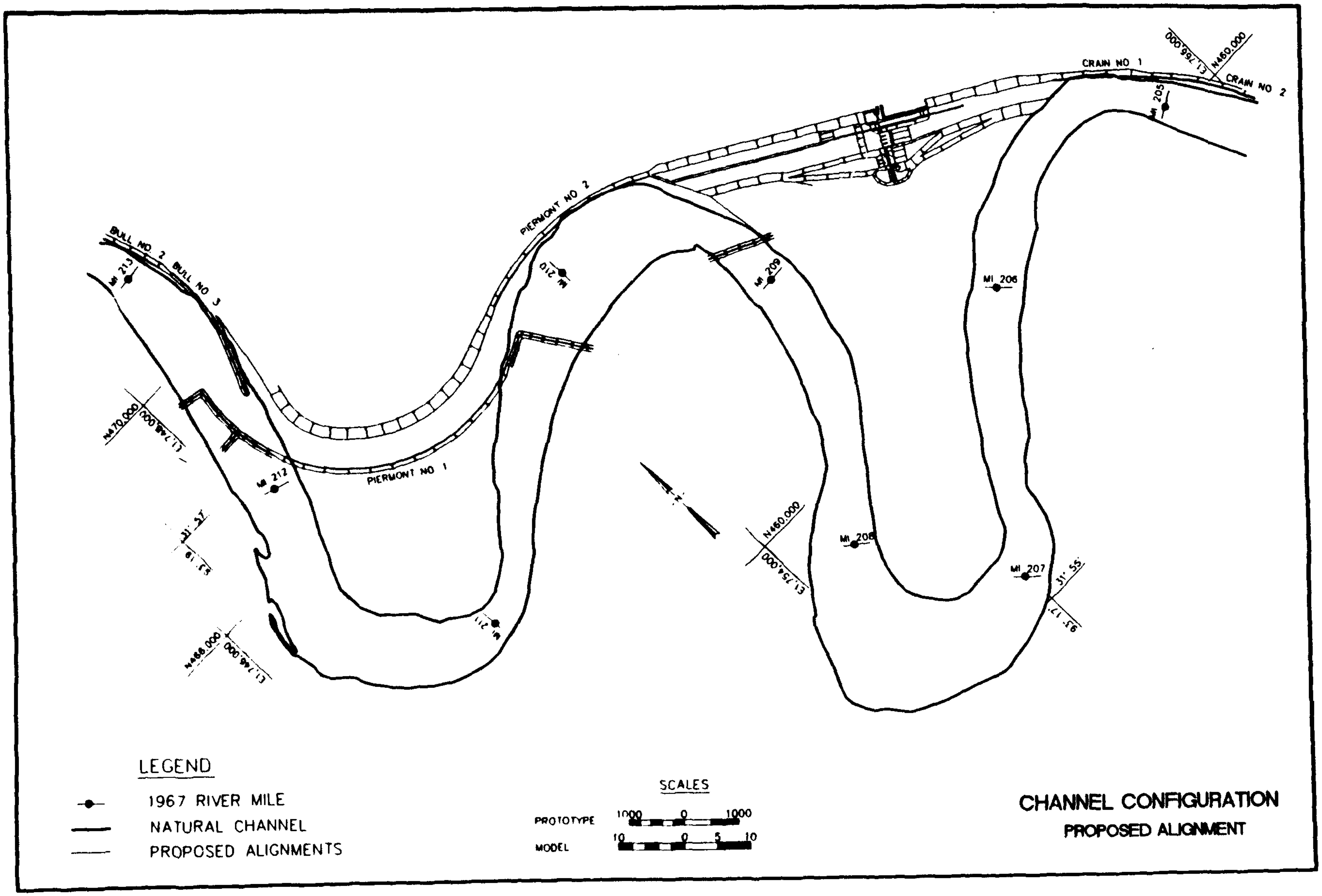
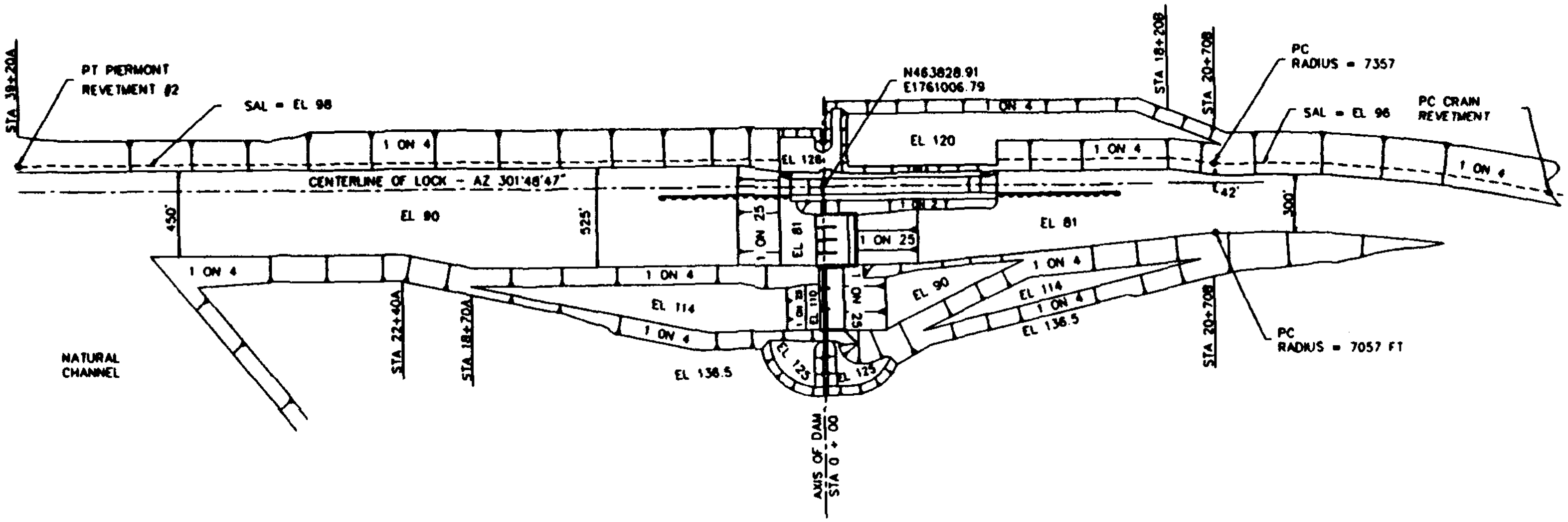

CRAIN NO 1
CRAIN NO 2
N460,000
E1,766,000
MI 205
MI 206
MI 207
MI 208
MI 209
MI 210
MI 211
MI 212
MI 213
PIERMONT NO 1
PIERMONT NO 2
BULL NO 2
BULL NO 3
N470,000
E1,748,000
N468,000
E1,746,000
N460,000
E1,754,000
31° 57'
31° 55'
LEGEND
1967 RIVER MILE
NATURAL CHANNEL
PROPOSED ALIGNMENTS
SCALES
PROTOTYPE 1000 0 1000
MODEL 10 0 5 10
CHANNEL CONFIGURATION
PROPOSED ALIGNMENT

PLATE 14



SCALES

NOTE: ALL ELEVATIONS ARE IN FEET REFERRED TO NGVD

LOCK AND DAM
DETAILED PLAN VIEW

PLATE 15

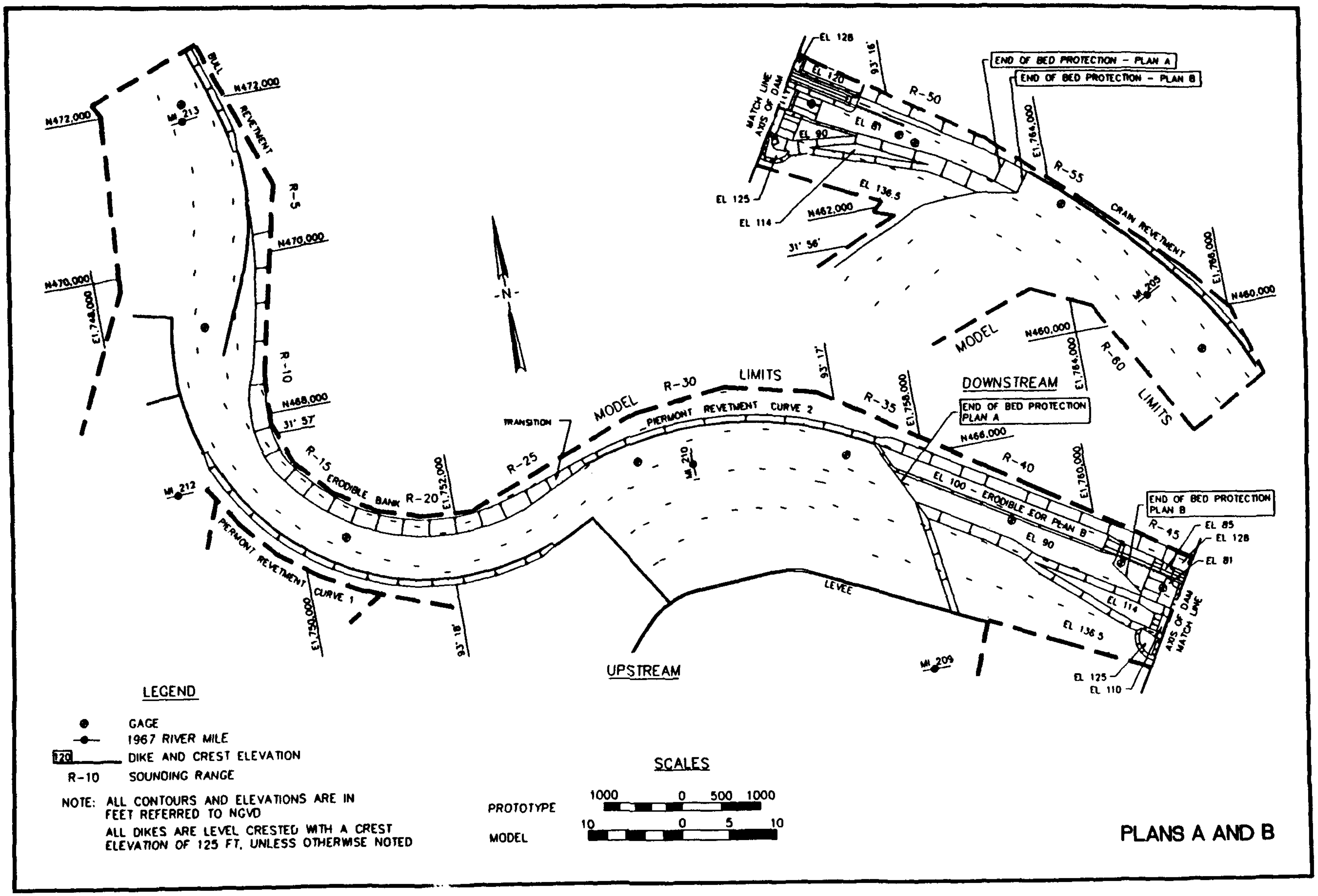
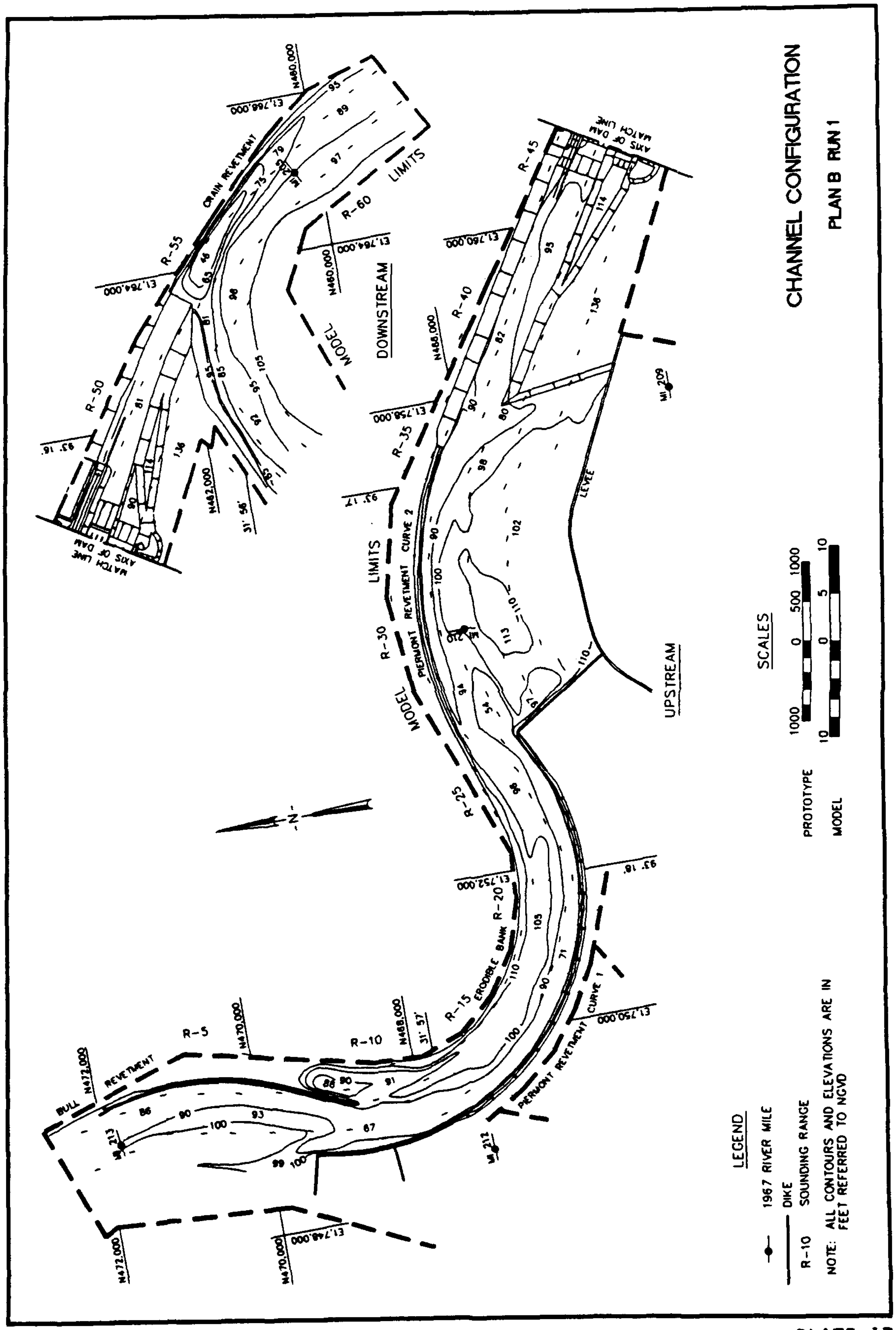

PLANS A AND B
LEGEND
GAGE
1967 RIVER MILE
120 DIKE AND CREST ELEVATION
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
ALL DIKES ARE LEVEL CRESTED WITH A CREST ELEVATION OF 125 FT, UNLESS OTHERWISE NOTED
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
UPSTREAM
DOWNSTREAM
MODEL LIMITS
END OF BED PROTECTION - PLAN A
END OF BED PROTECTION - PLAN B
END OF BED PROTECTION PLAN A
END OF BED PROTECTION PLAN B
BULL REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
CRAIN REVETMENT
ERODIBLE BANK
TRANSITION
LEVEE
EL 100 - ERODIBLE FOR PLAN B
MATCH LINE
AXIS OF DAM
R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
R-55
R-60
N472,000
N470,000
N468,000
N466,000
N462,000
N460,000
E1,748,000
E1,750,000
E1,752,000
E1,756,000
E1,760,000
E1,764,000
E1,766,000
31° 57'
31° 56'
93° 18'
93° 17'
93° 16'
EL 128
EL 120
EL 81
EL 90
EL 125
EL 114
EL 136.5
EL 85
EL 110
-N-
MI 213
MI 212
MI 210
MI 209

PLATE 16


CHANNEL CONFIGURATION
PLAN B RUN 1
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
UPSTREAM
DOWNSTREAM
MODEL LIMITS
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
BULL REVETMENT
DRAIN REVETMENT
LEVEE
AXIS OF DAM
MATCH LINE
R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
R-55
R-60
MI 209
MI 210
MI 212
MI 213
N472,000
N470,000
N468,000
N466,000
N462,000
N460,000
E1,748,000
E1,750,000
E1,752,000
E1,756,000
E1,760,000
E1,764,000
E1,766,000
31° 57'
31° 56'
93° 16'
93° 17'
93° 18'
N

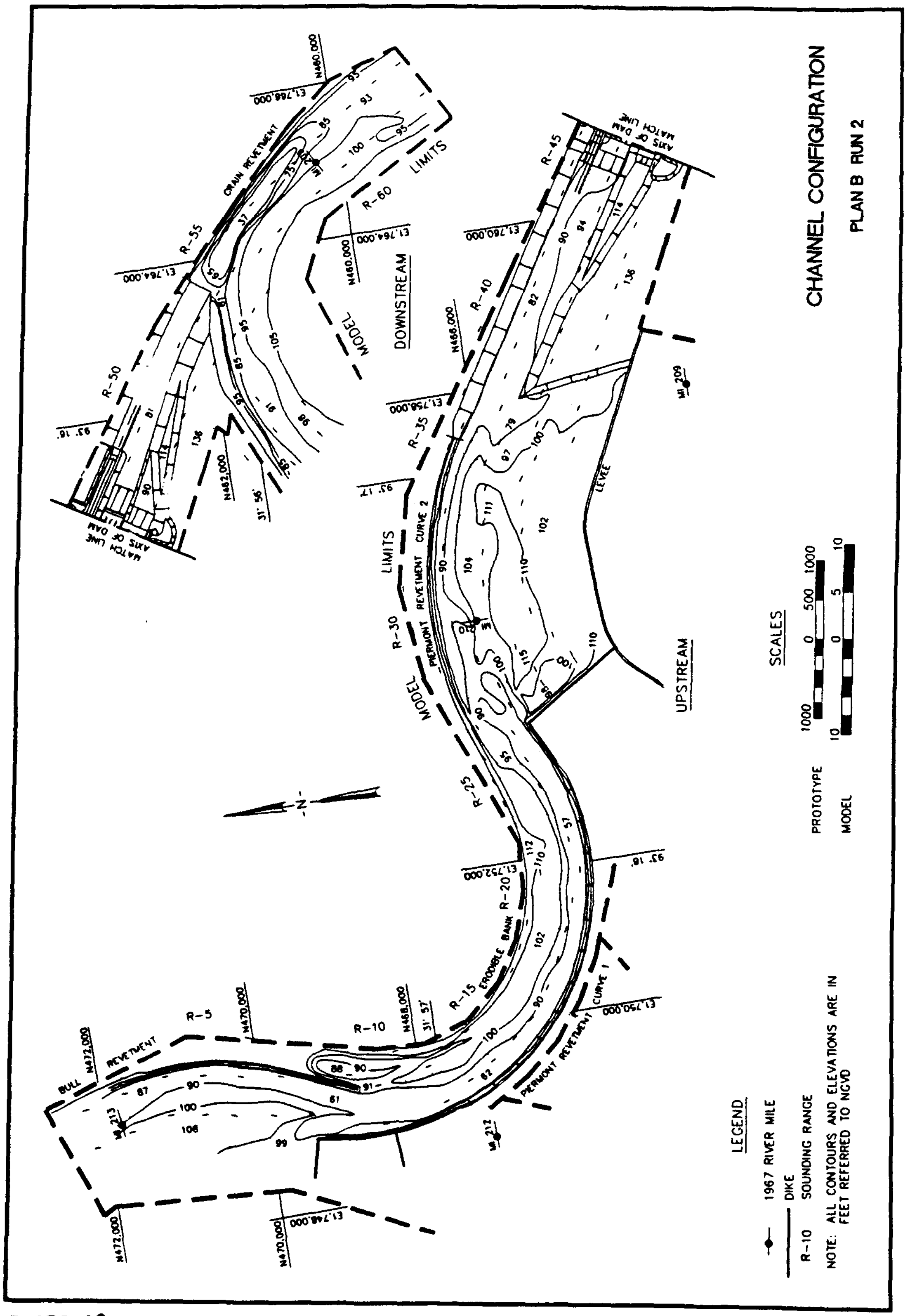
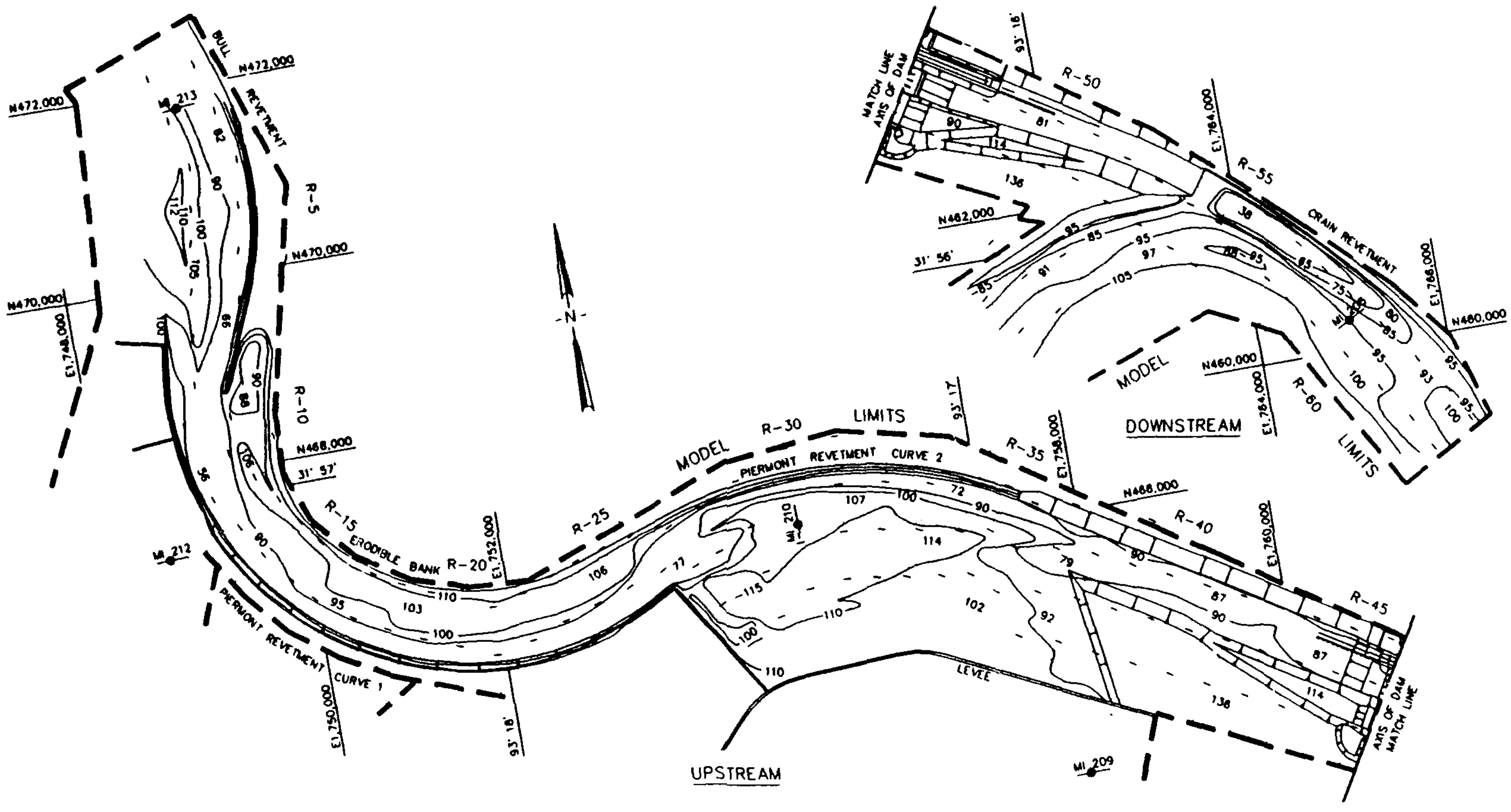

CHANNEL CONFIGURATION
PLAN B RUN 2
SCALES
PROTOTYPE
MODEL
1000 0 500 1000
10 0 5 10
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
UPSTREAM
DOWNSTREAM
MODEL LIMITS
MATCH LINE
AXIS OF DAM
LEVEE
BULL REVETMENT
PIERMONT REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
CRAIN REVETMENT
R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
R-55
R-60
N472,000
N470,000
N468,000
N466,000
N462,000
N460,000
E1,748,000
E1,750,000
E1,752,000
E1,758,000
E1,760,000
E1,764,000
E1,768,000
31° 57'
31° 56'
93° 18'
93° 17'
MI 213
MI 212
MI 210
MI 209
N

PLATE 18



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN B RUN 3

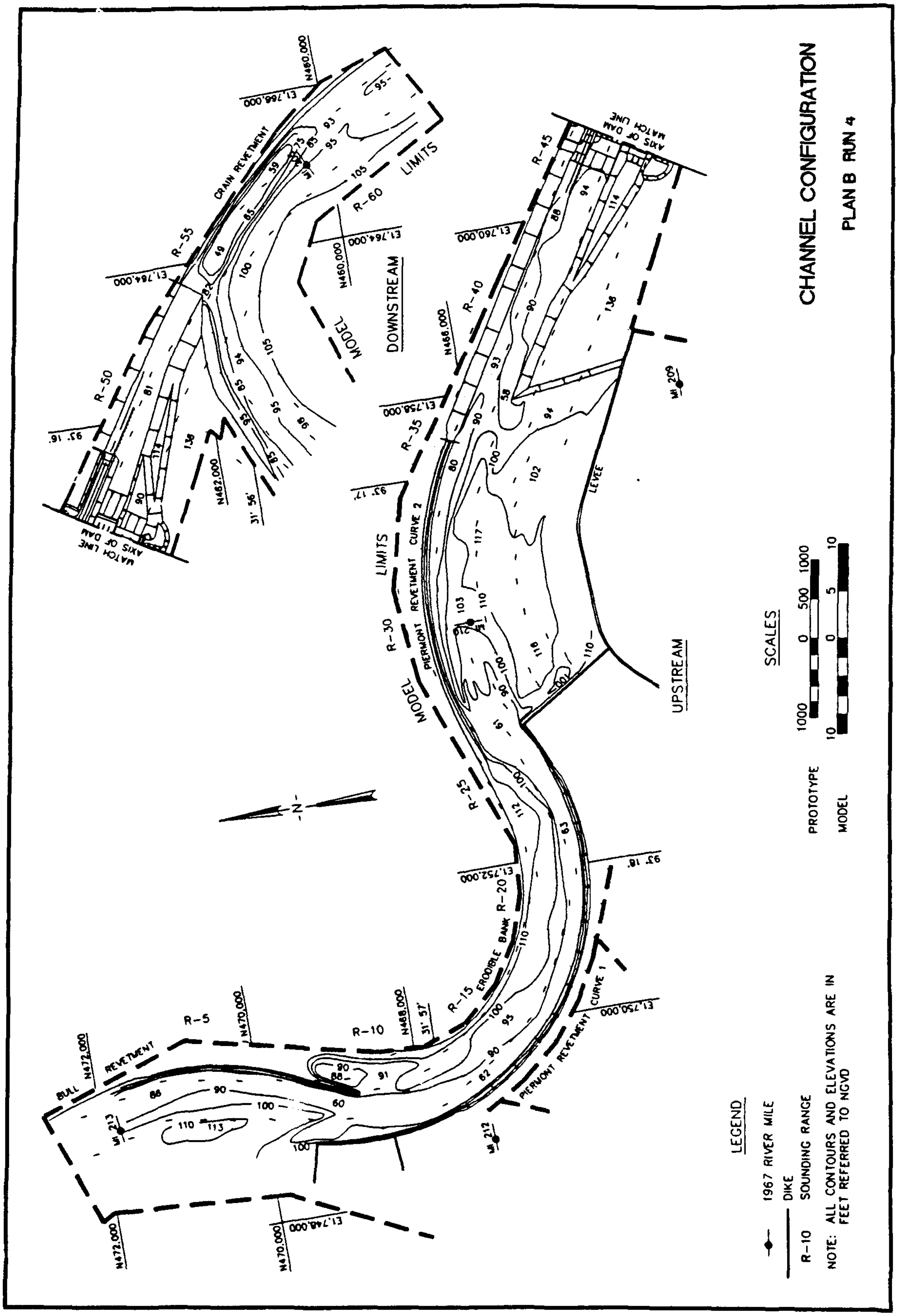
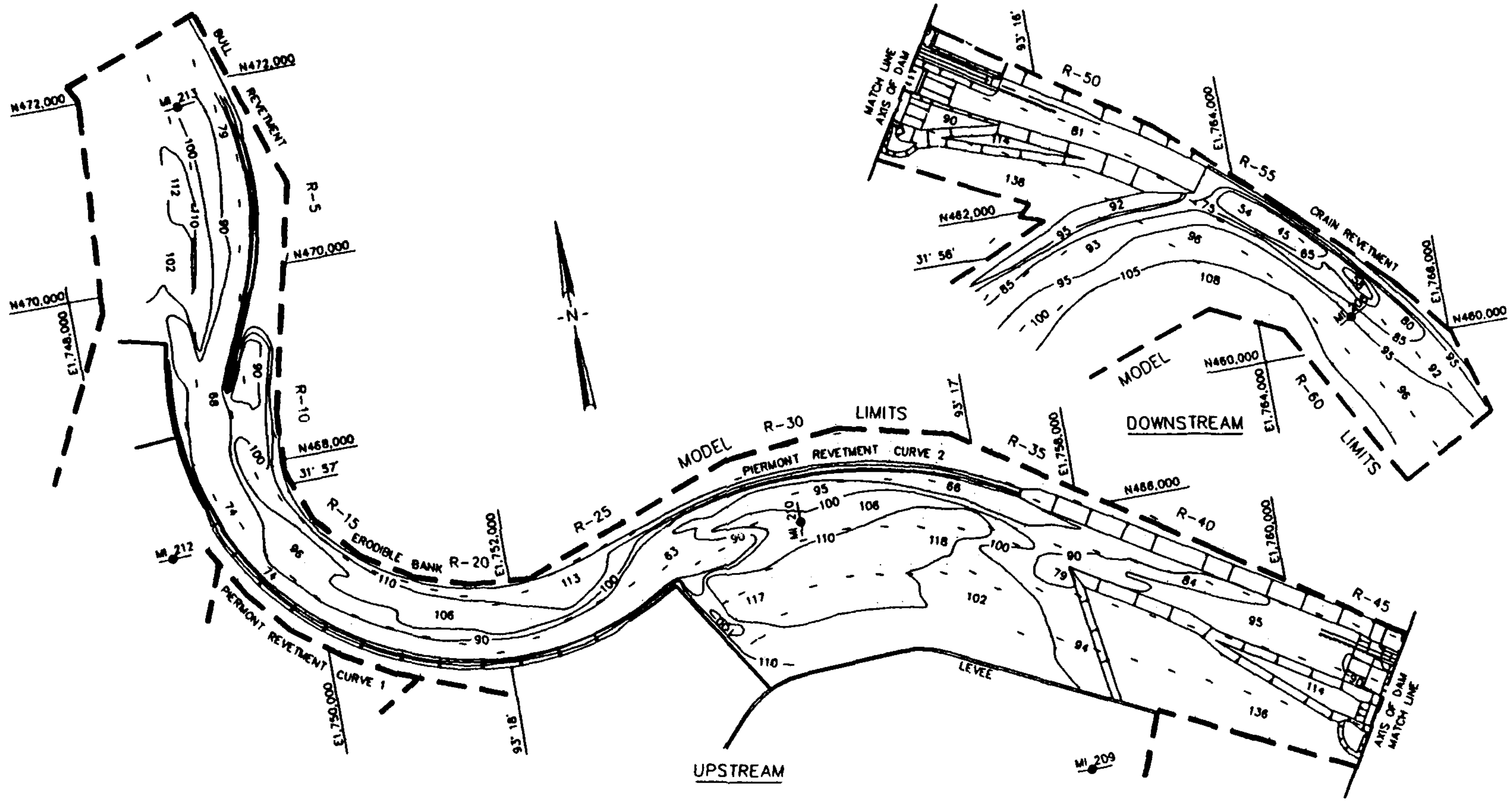

CHANNEL CONFIGURATION
PLAN B RUN 4
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
UPSTREAM
DOWNSTREAM
MODEL LIMITS
LEVEE
BULL REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
CRAIN REVETMENT
AXIS OF DAM
MATCH LINE
-N-

PLATE 20



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

SCALES

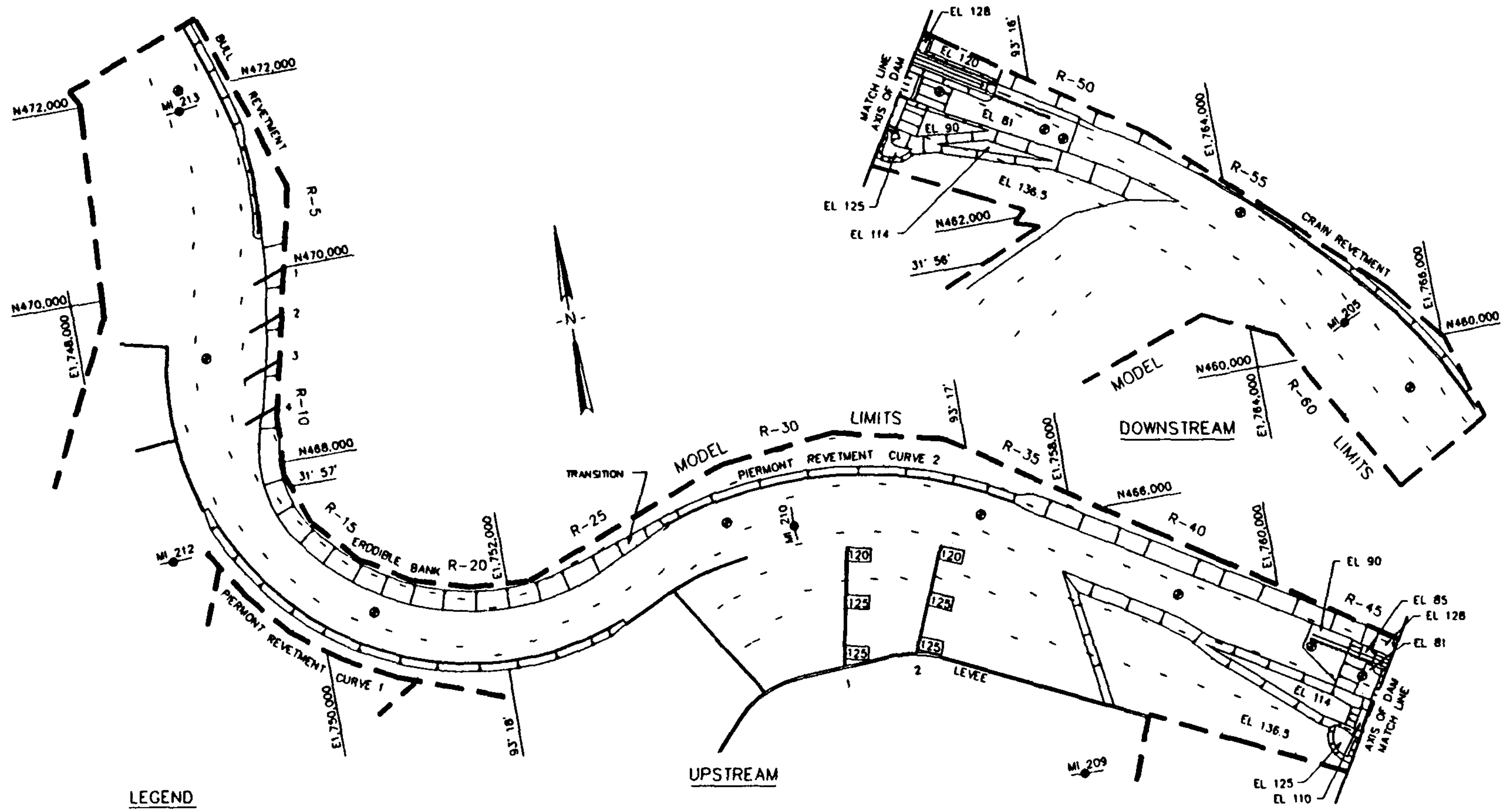
PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN B RUN 5

PLATE 21



LEGEND

- GAGE
- 1967 RIVER MILE
- 120 DIKE AND CREST ELEVATION
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

ALL DIKES ARE LEVEL CRESTED WITH A CREST ELEVATION OF 125 FT, UNLESS OTHERWISE NOTED

SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

PLAN C

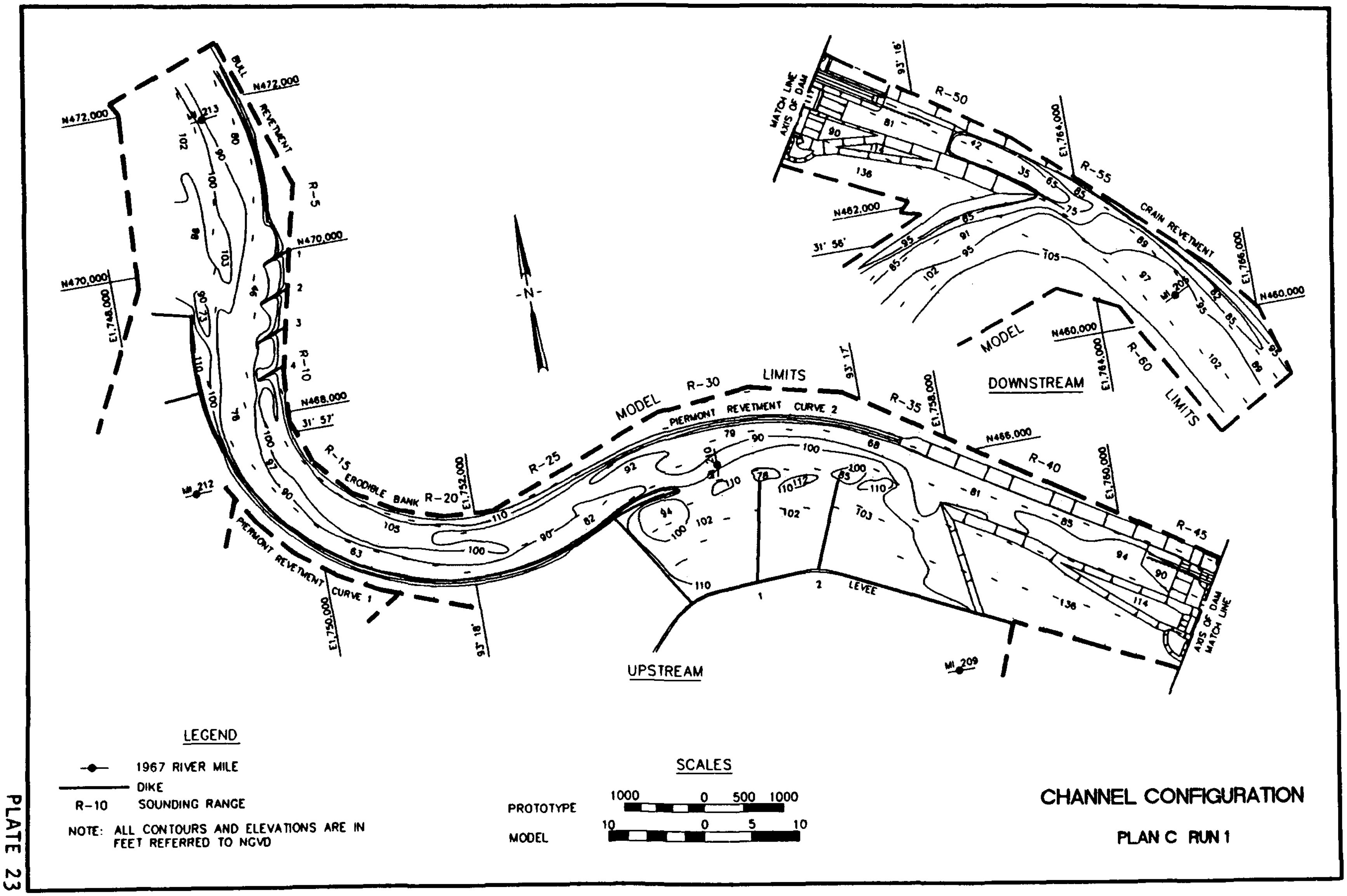

BULL REVETMENT
R-5
R-10
R-15
ERODIBLE BANK
R-20
R-25
R-30
R-35
R-40
R-45
R-50
R-55
R-60
CRAIN REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
MODEL LIMITS
UPSTREAM
DOWNSTREAM
LEVEE
AXIS OF DAM
MATCH LINE
N472,000
N470,000
N468,000
N466,000
N462,000
N460,000
E1,748,000
E1,750,000
E1,752,000
E1,758,000
E1,760,000
E1,764,000
E1,766,000
31° 57'
31° 56'
93° 18'
93° 17'
93° 16'
MI 213
MI 212
MI 209
MI 208
-N-
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
CHANNEL CONFIGURATION
PLAN C RUN 1

PLATE 23

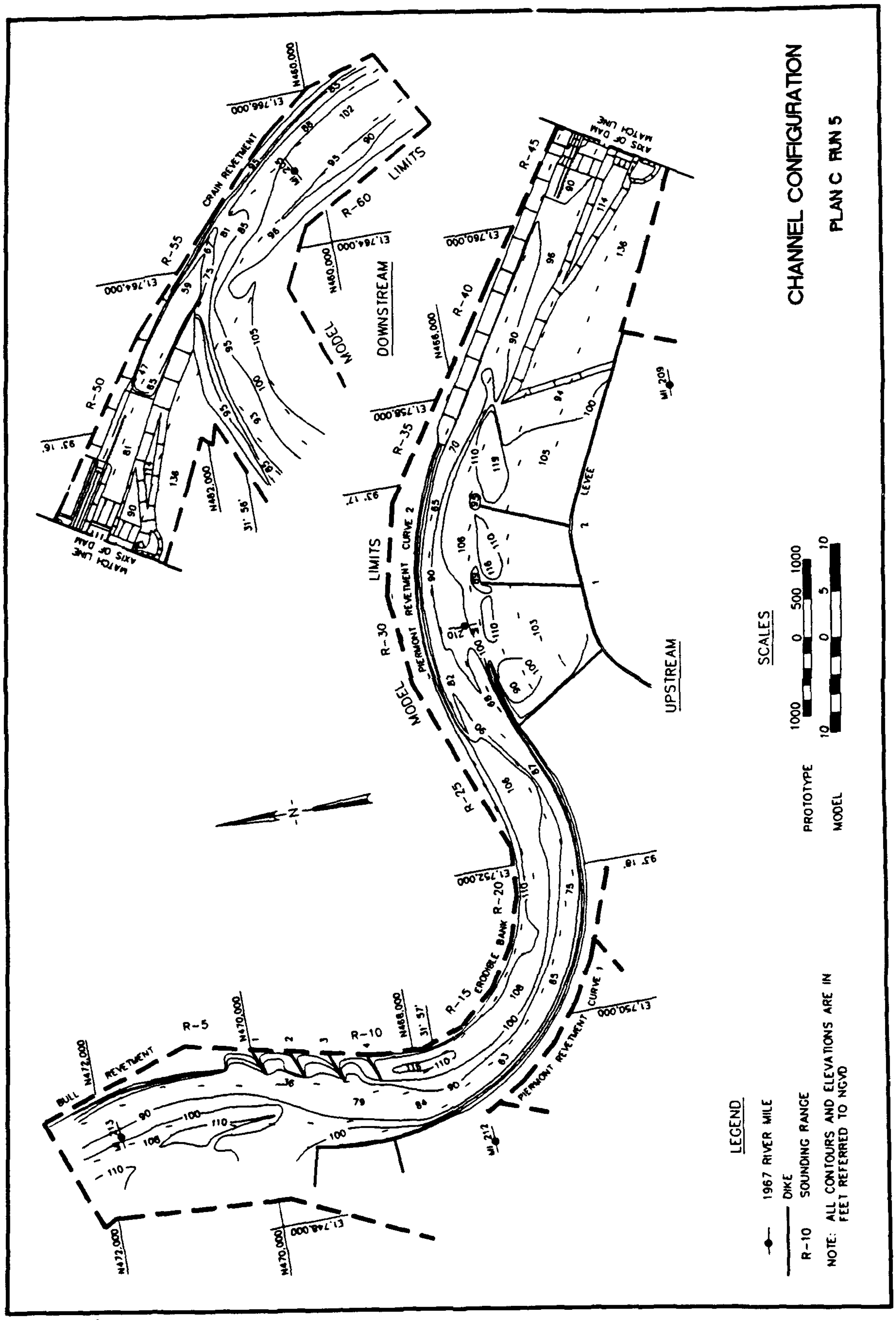
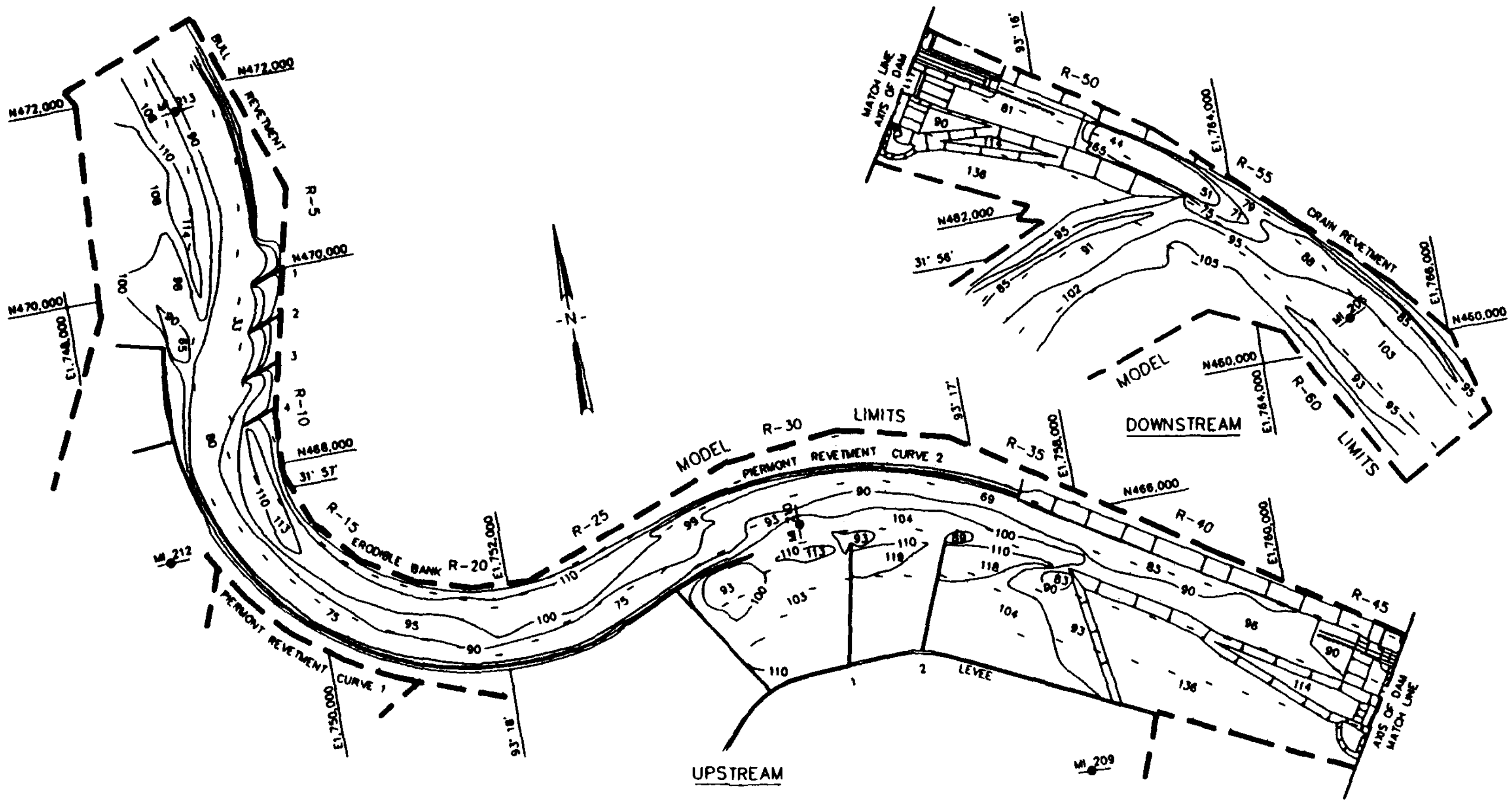

CHANNEL CONFIGURATION
PLAN C RUN 5
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
MODEL LIMITS
UPSTREAM
MODEL LIMITS
DOWNSTREAM
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
BULL REVETMENT
CRAIN REVETMENT
LEVEE
AXIS OF DAM
MATCH LINE
MI 212
MI 209

PLATE 24



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

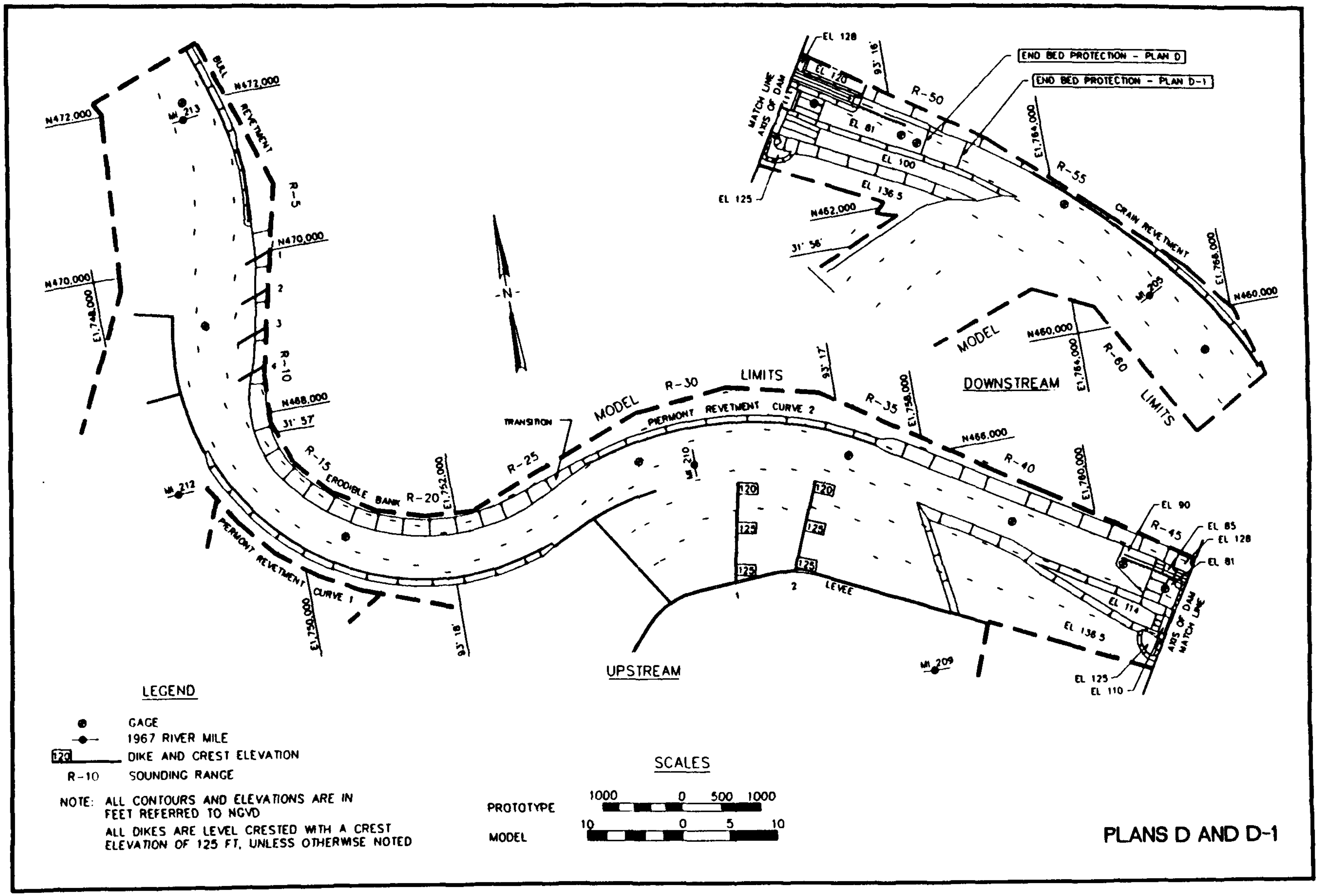
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

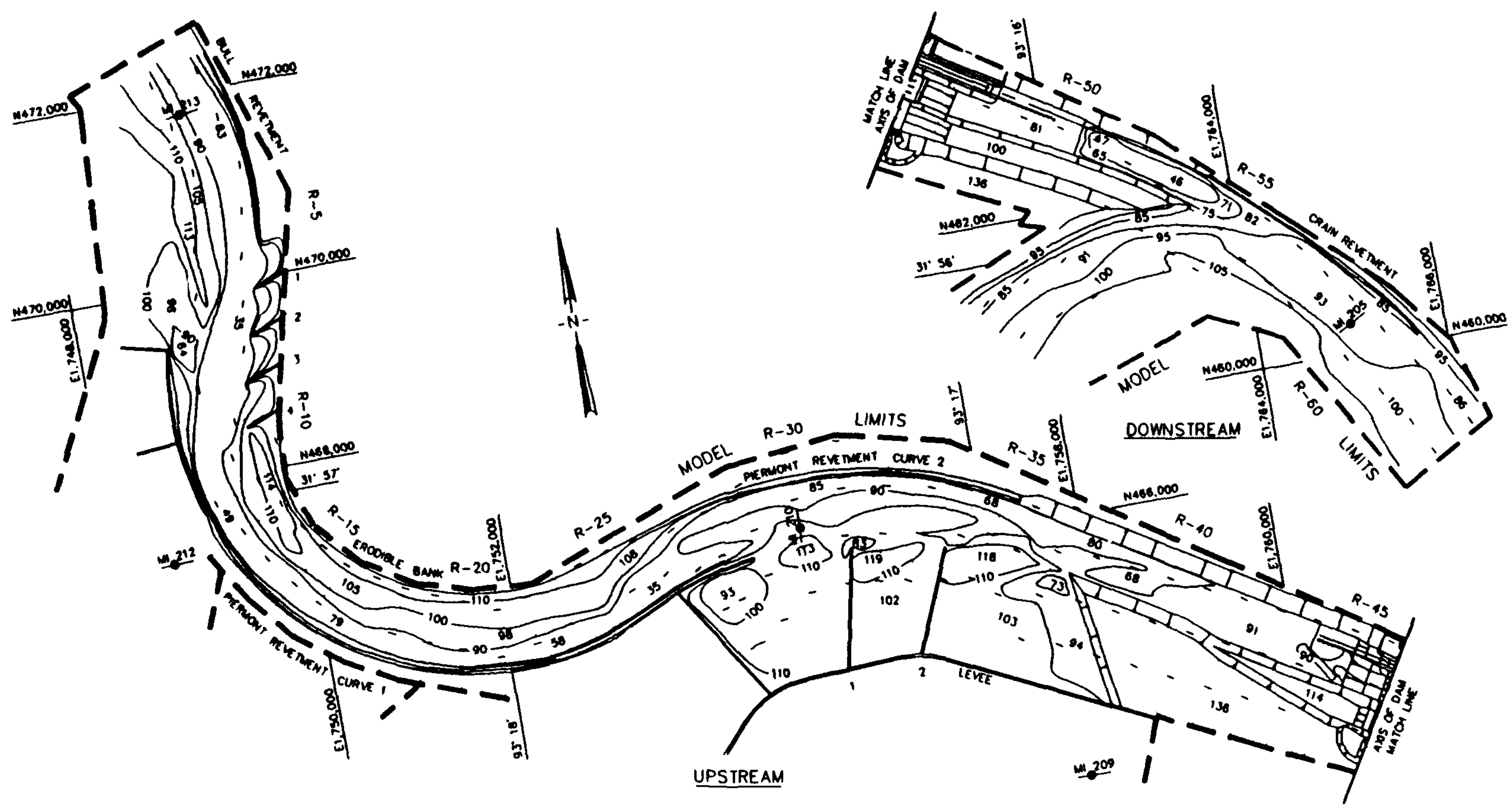
CHANNEL CONFIGURATION

PLAN C RUN 10



PLANS D AND D-1

PLATE 26



LEGEND

1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

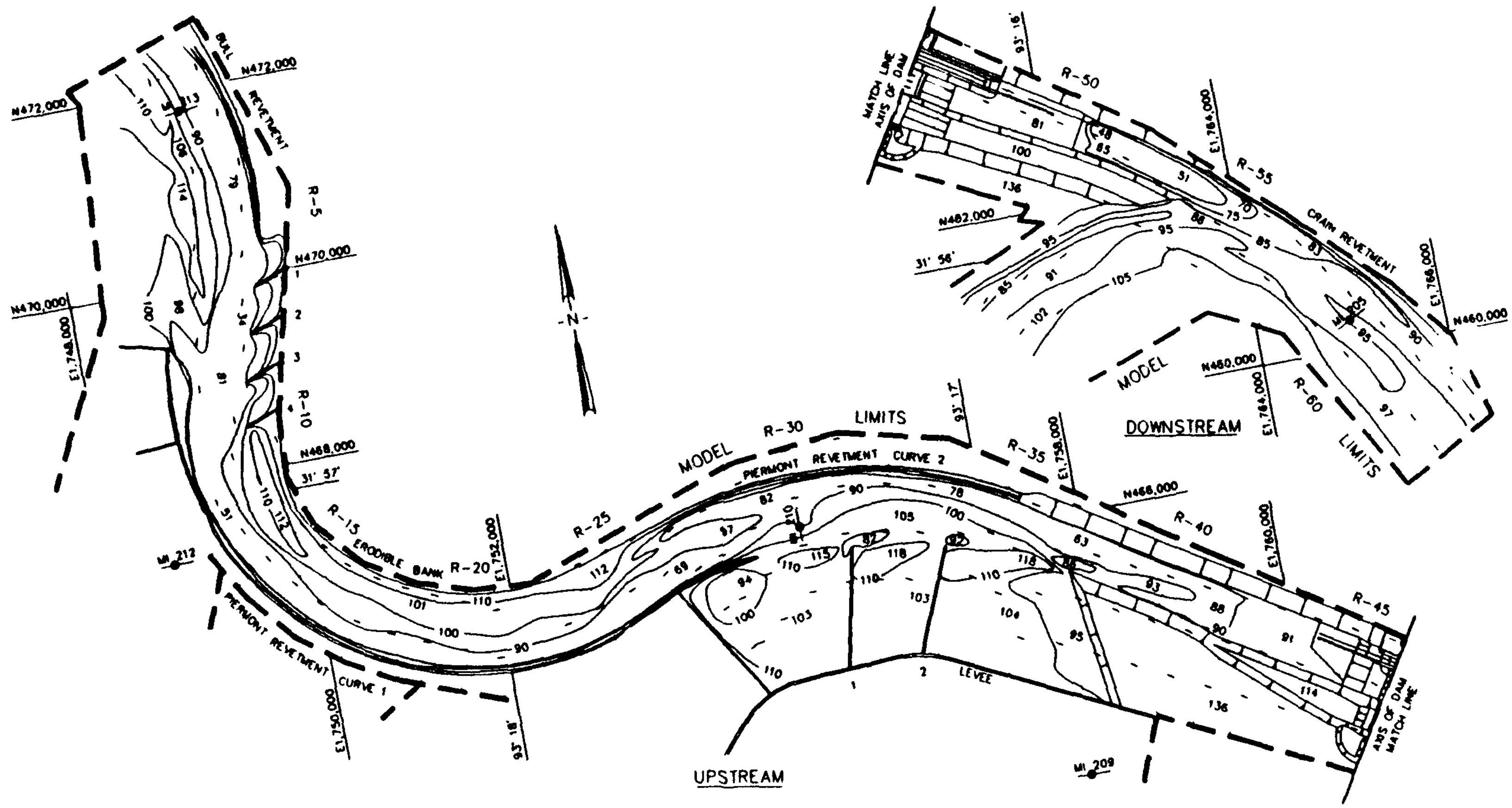
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D RUN 1



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

SCALES

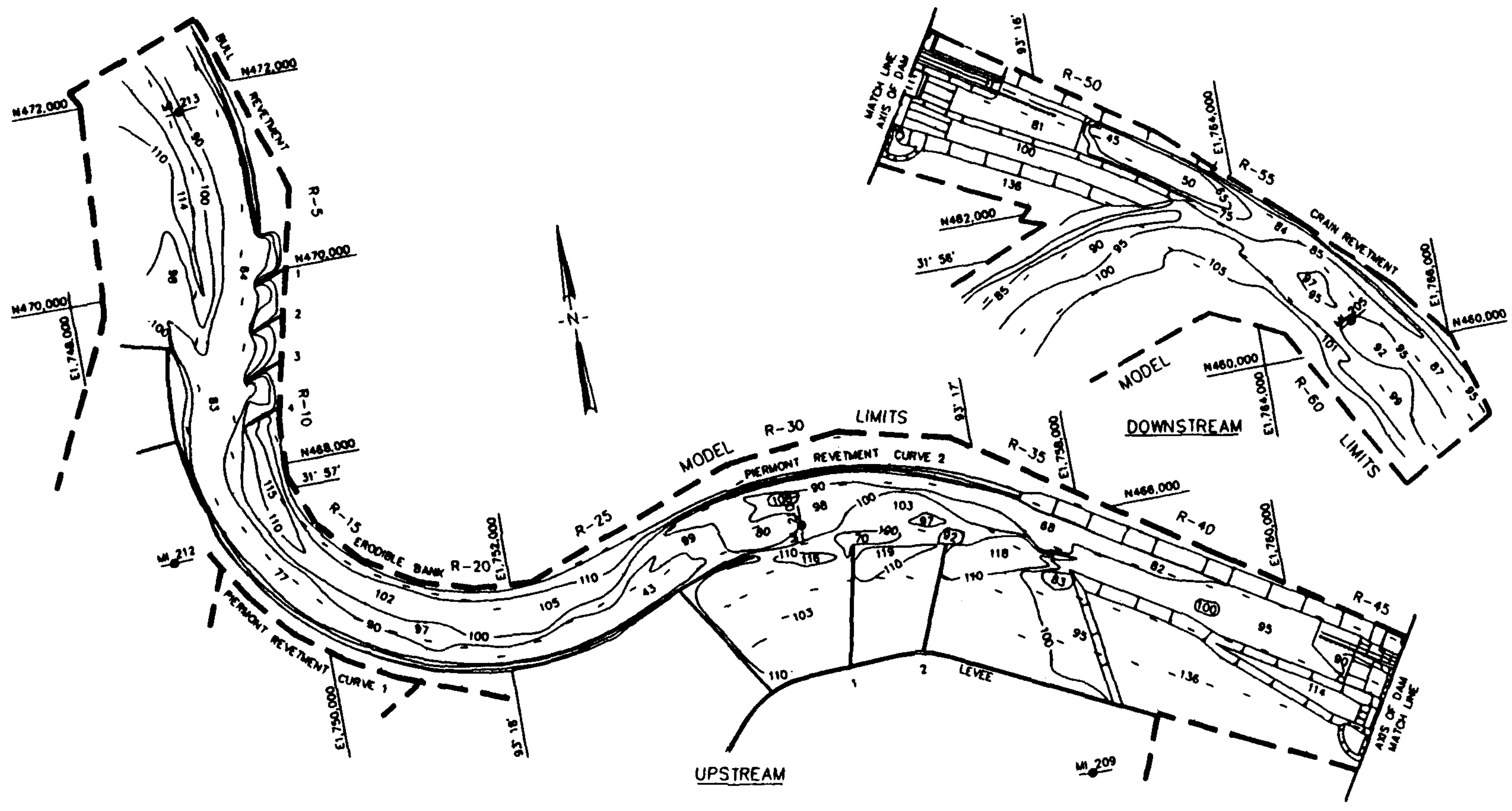
PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D RUN 2

PLATE 28



LEGEND

1967 RIVER MILE

DIKE

R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

SCALES

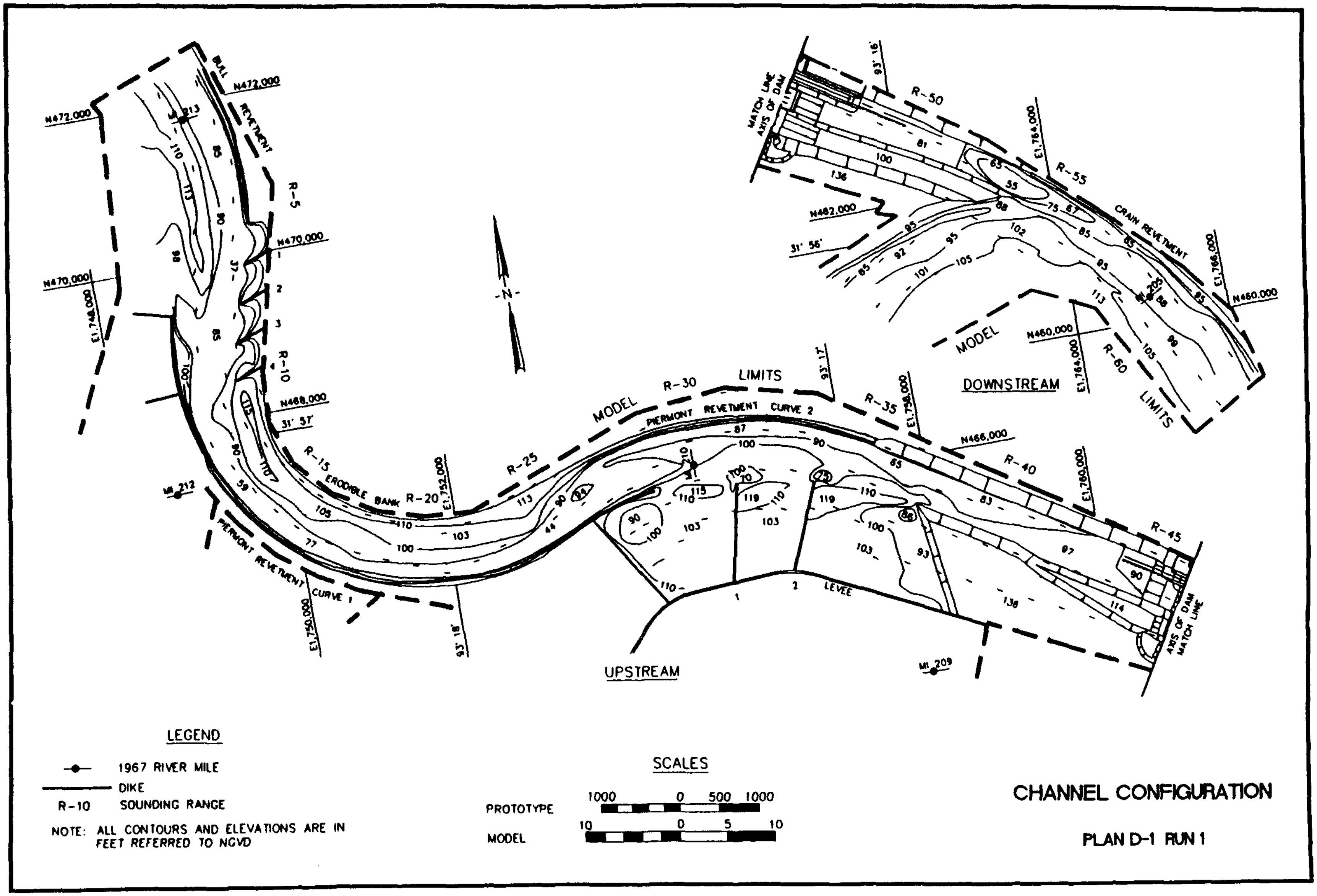
PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

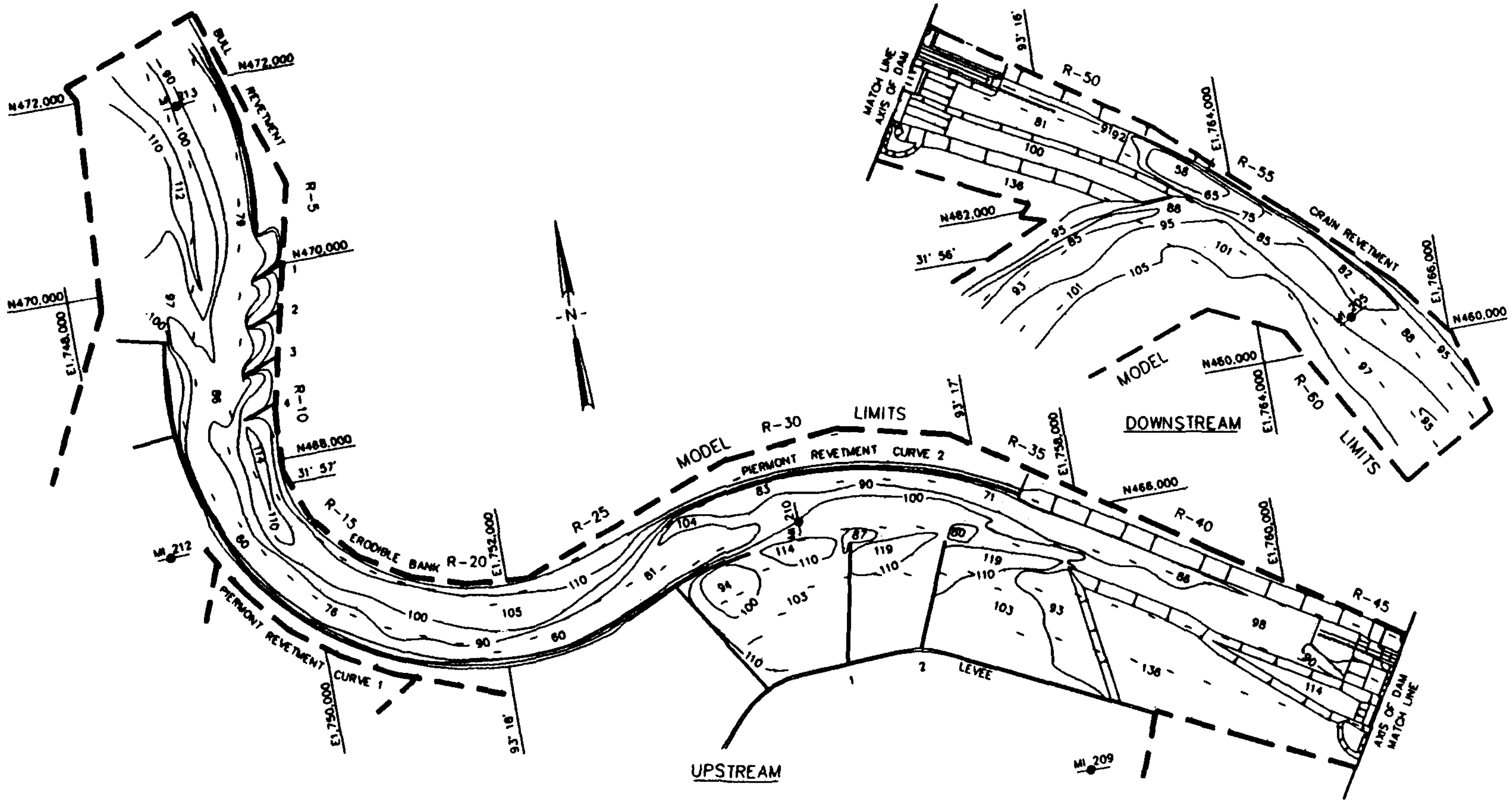
PLAN D RUN 3

PLATE 29



CHANNEL CONFIGURATION

PLAN D-1 RUN 1



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

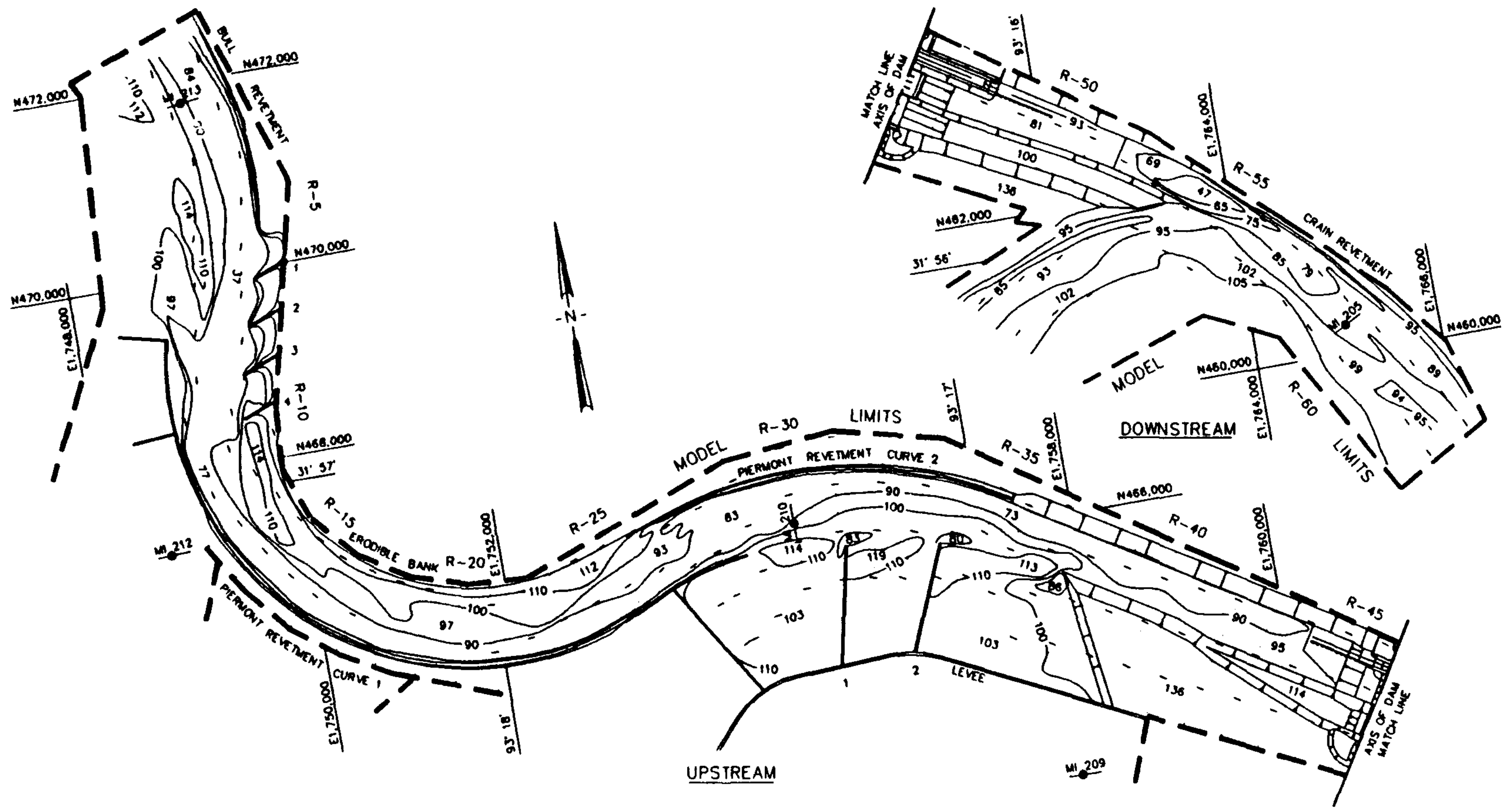
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D-1 RUN 2



LEGEND

—•—	1967 RIVER MILE
———	DIKE
R-10	SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

SCALES

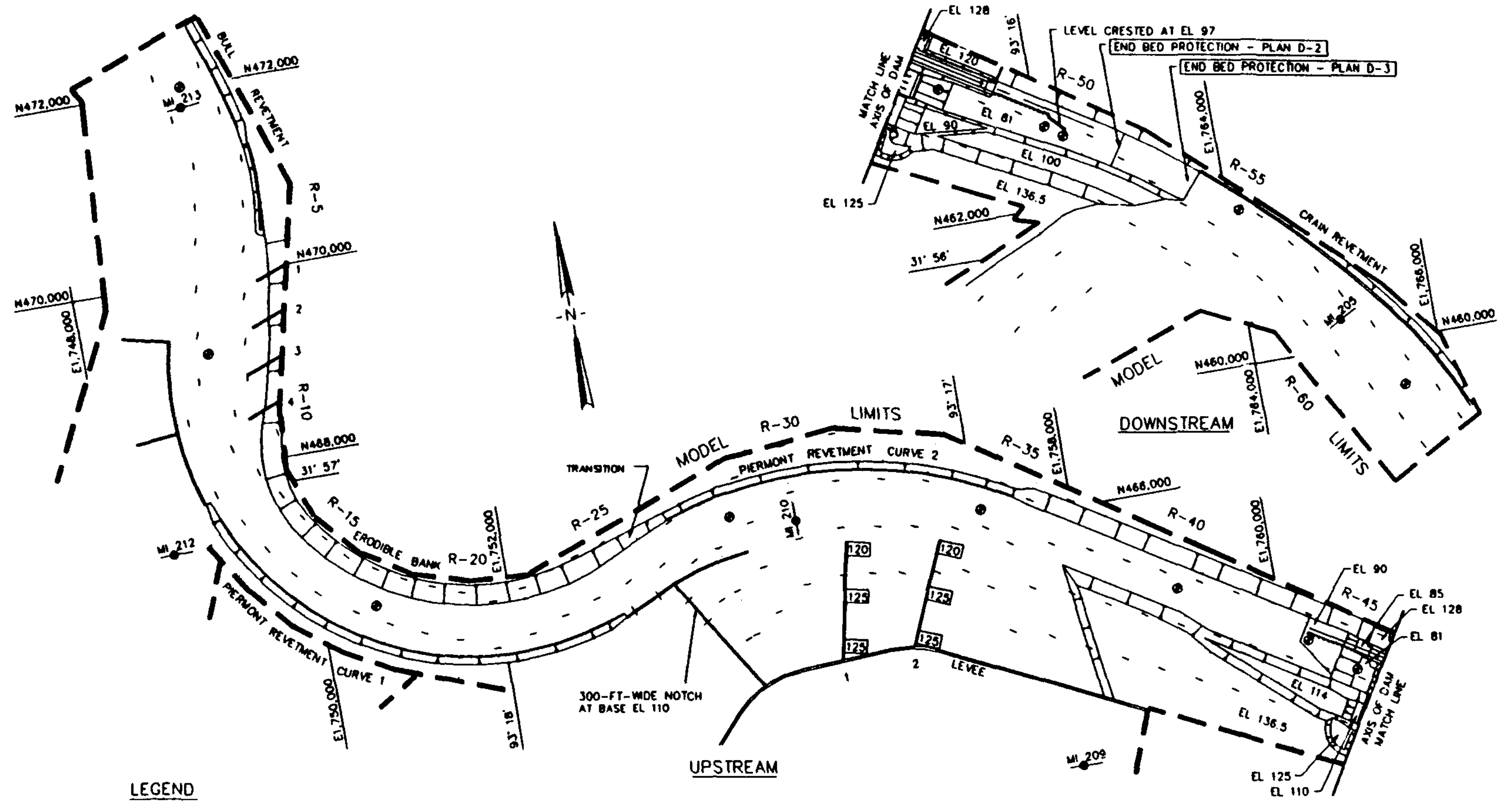
PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D-1 RUN 3

PLATE 32



LEGEND

- GAGE
- 1967 RIVER MILE
- 120 DIKE AND CREST ELEVATION
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
ALL DIKES ARE LEVEL CRESTED WITH A CREST ELEVATION OF 125 FT, UNLESS OTHERWISE NOTED

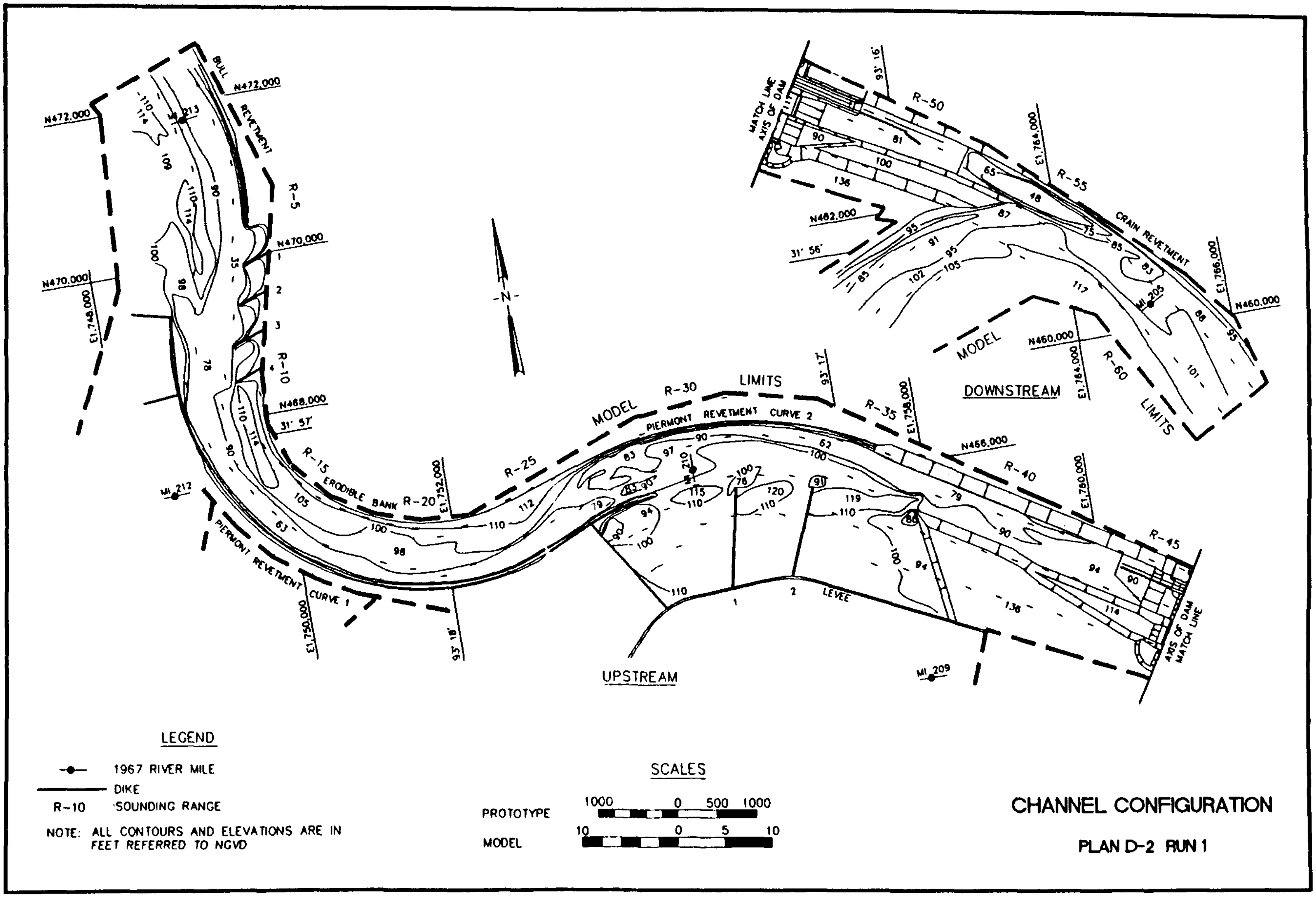
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

PLANS D-2 AND D-3

PLATE 33



CHANNEL CONFIGURATION

PLAN D-2 RUN 1

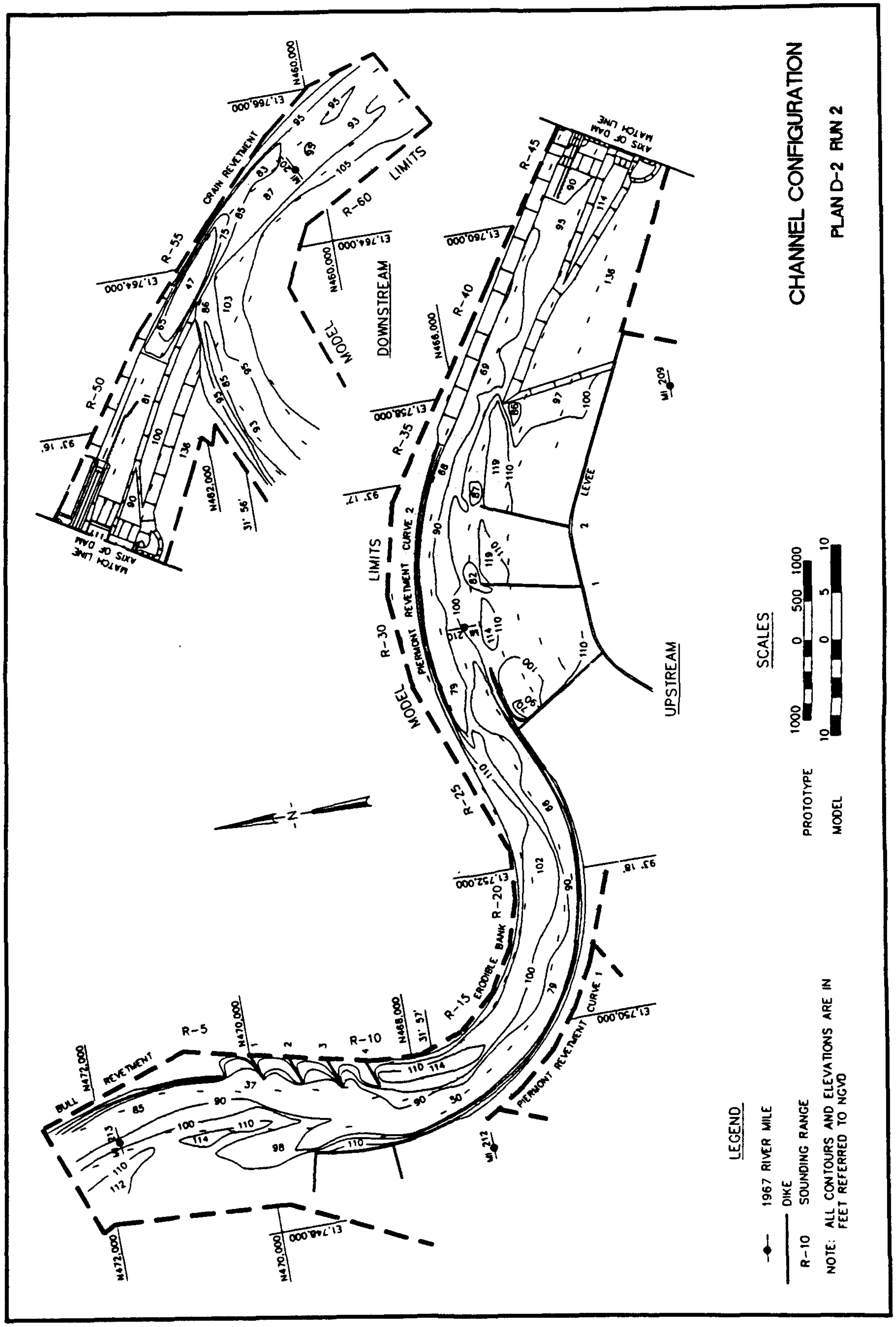
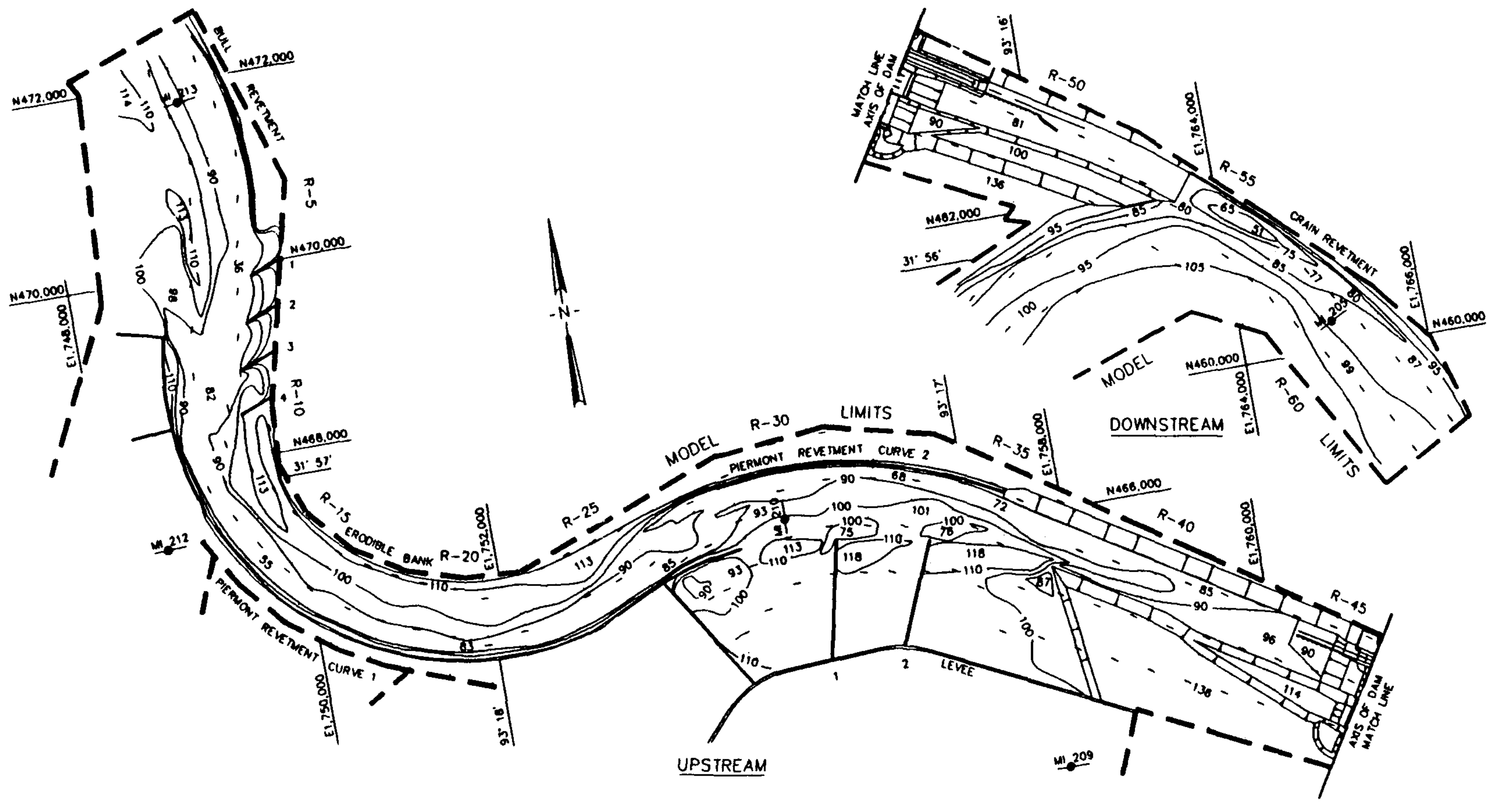

CHANNEL CONFIGURATION
PLAN D-2 RUN 2
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
MODEL LIMITS
DOWNSTREAM
UPSTREAM
PIERMONT REVETMENT
CURVE 1
CURVE 2
ERODIBLE BANK
BULL REVETMENT
CRAIN REVETMENT
AXIS OF DAM
MATCH LINE
LEVEE

PLATE 35



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

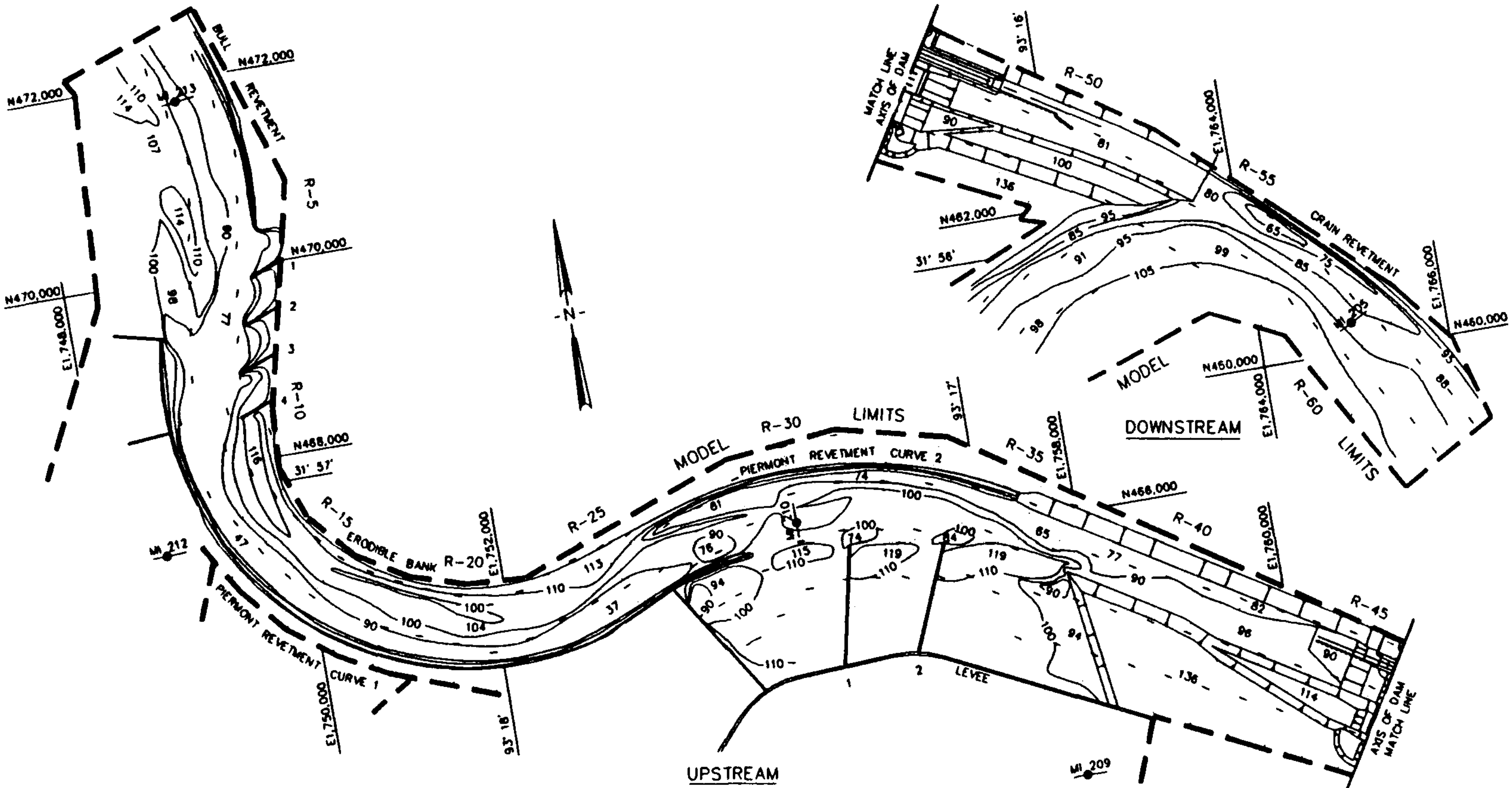
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D-3 RUN 1



LEGEND

1967 RIVER MILE

DIKE

R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

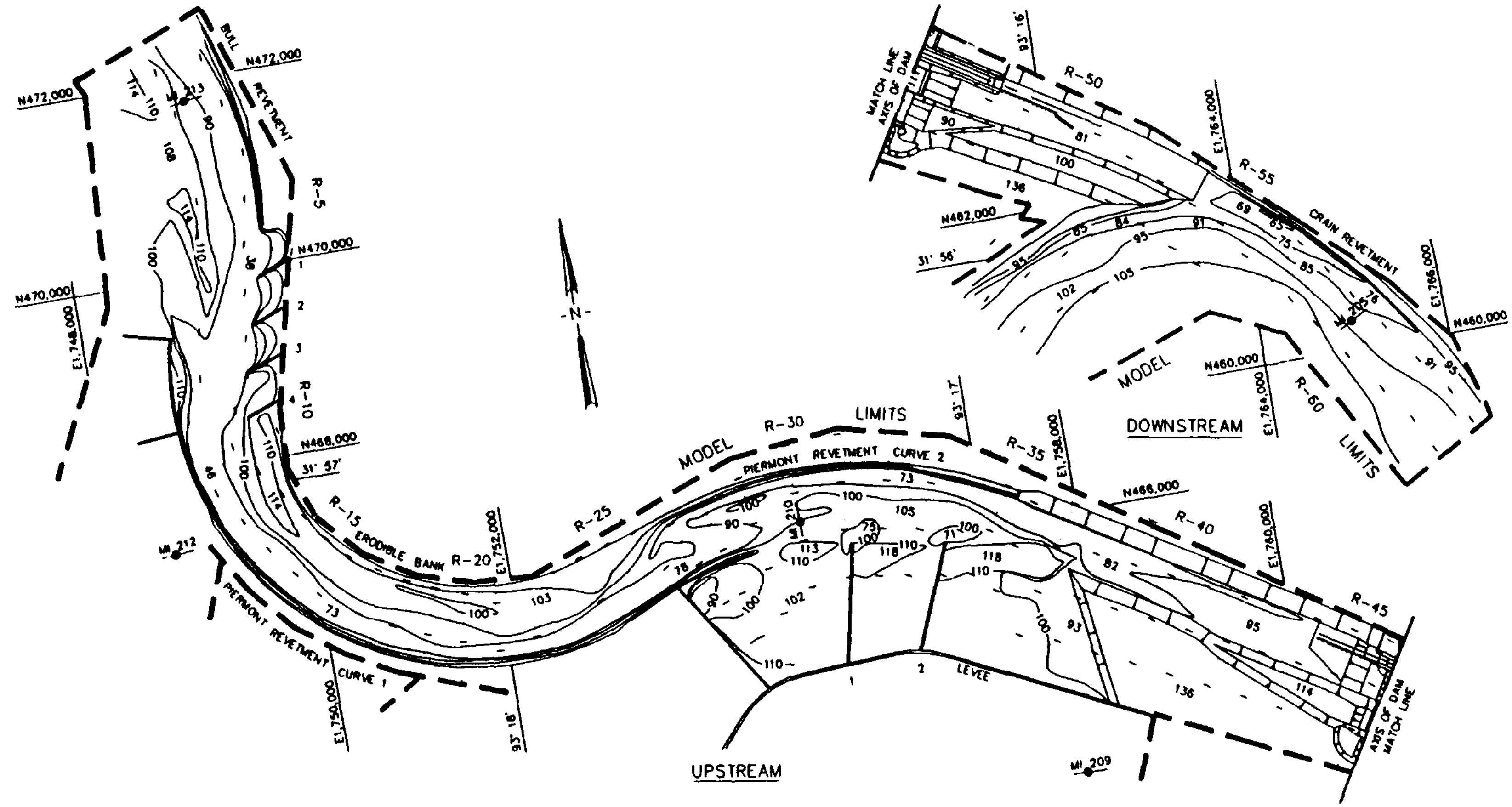
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D-3 RUN 2



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

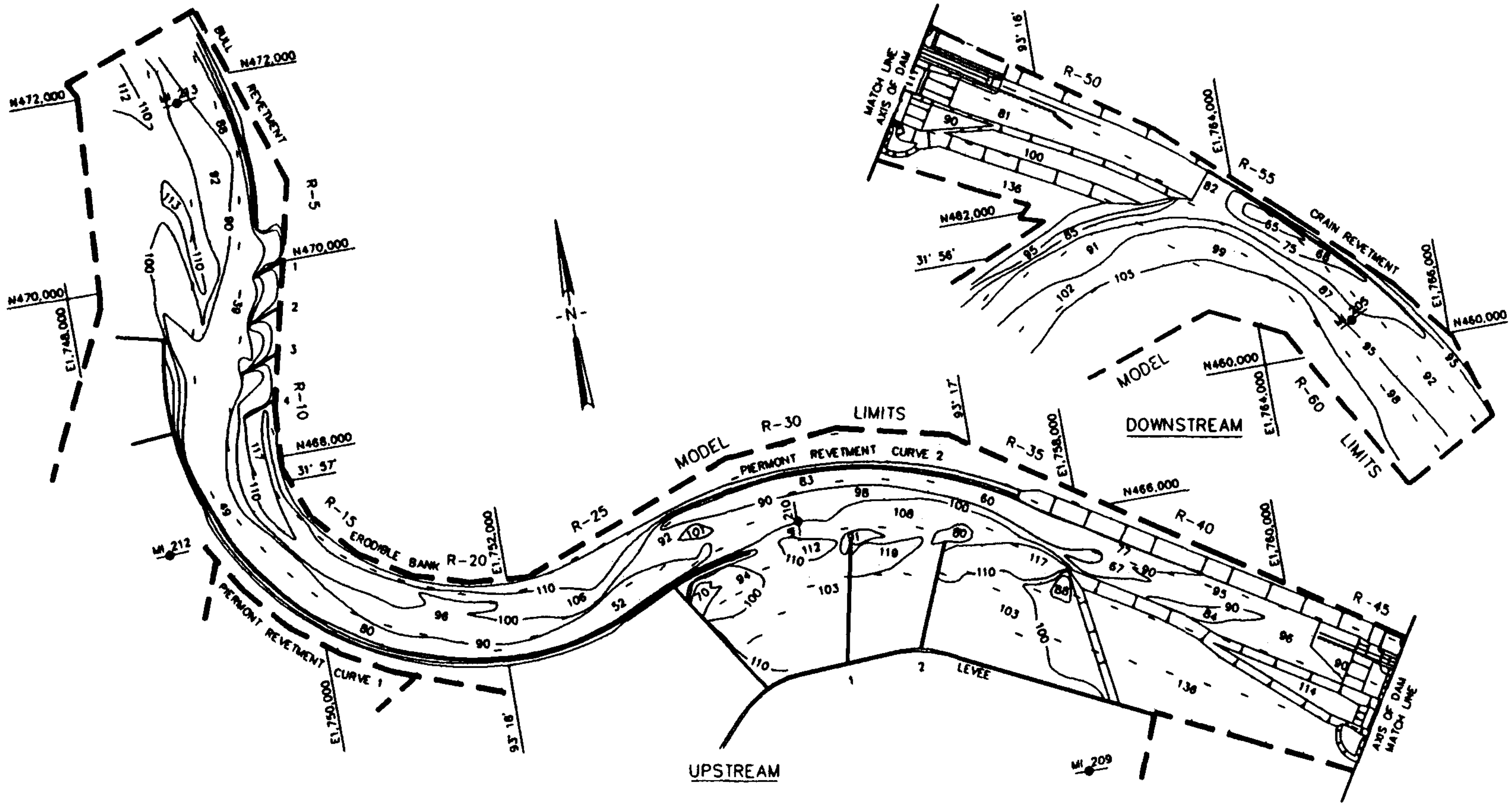
SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D-3 RUN 3



LEGEND

- 1967 RIVER MILE
- DIKE
- R-10 SOUNDING RANGE

NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD

SCALES

PROTOTYPE 1000 0 500 1000

MODEL 10 0 5 10

CHANNEL CONFIGURATION

PLAN D-3 RUN 4

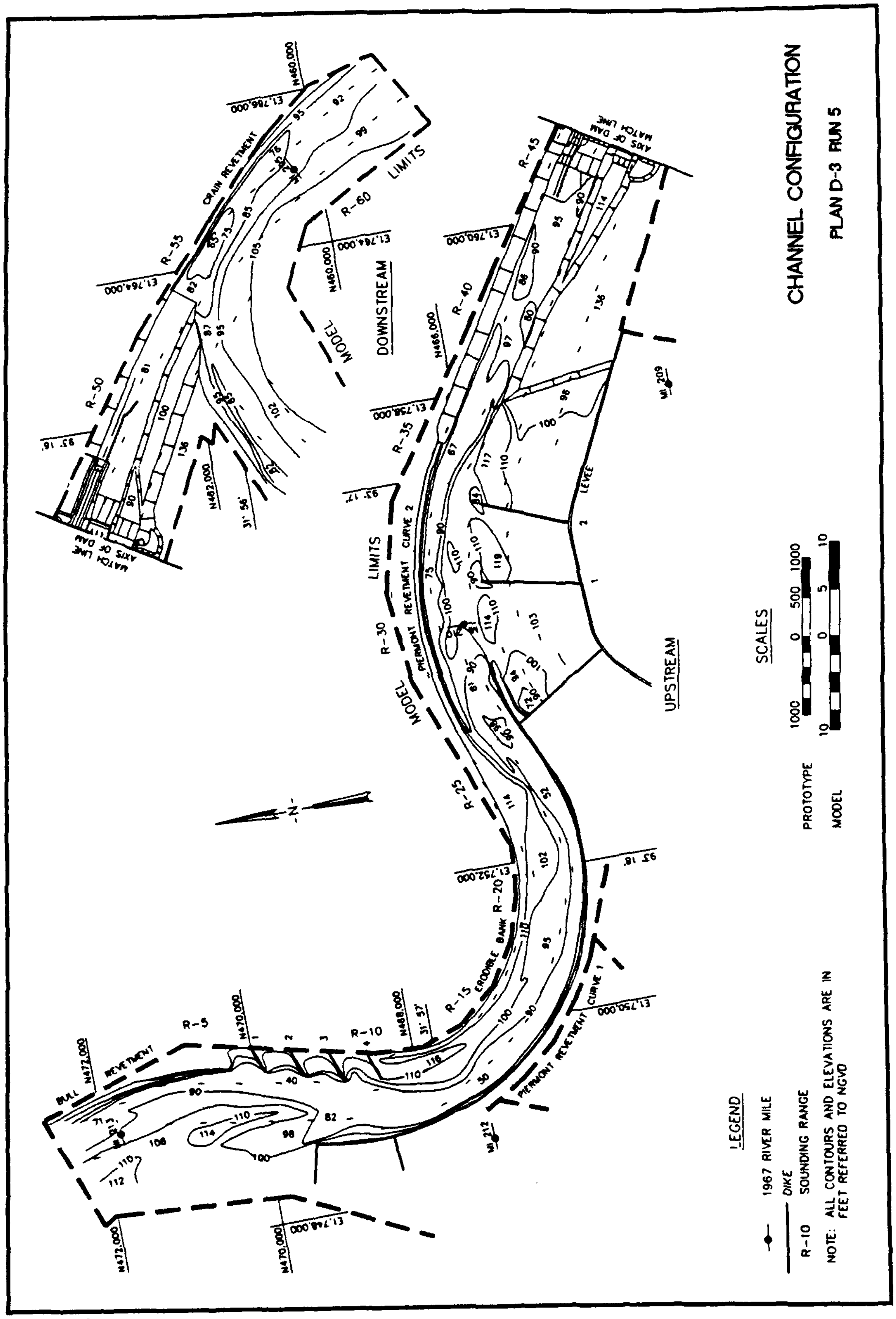
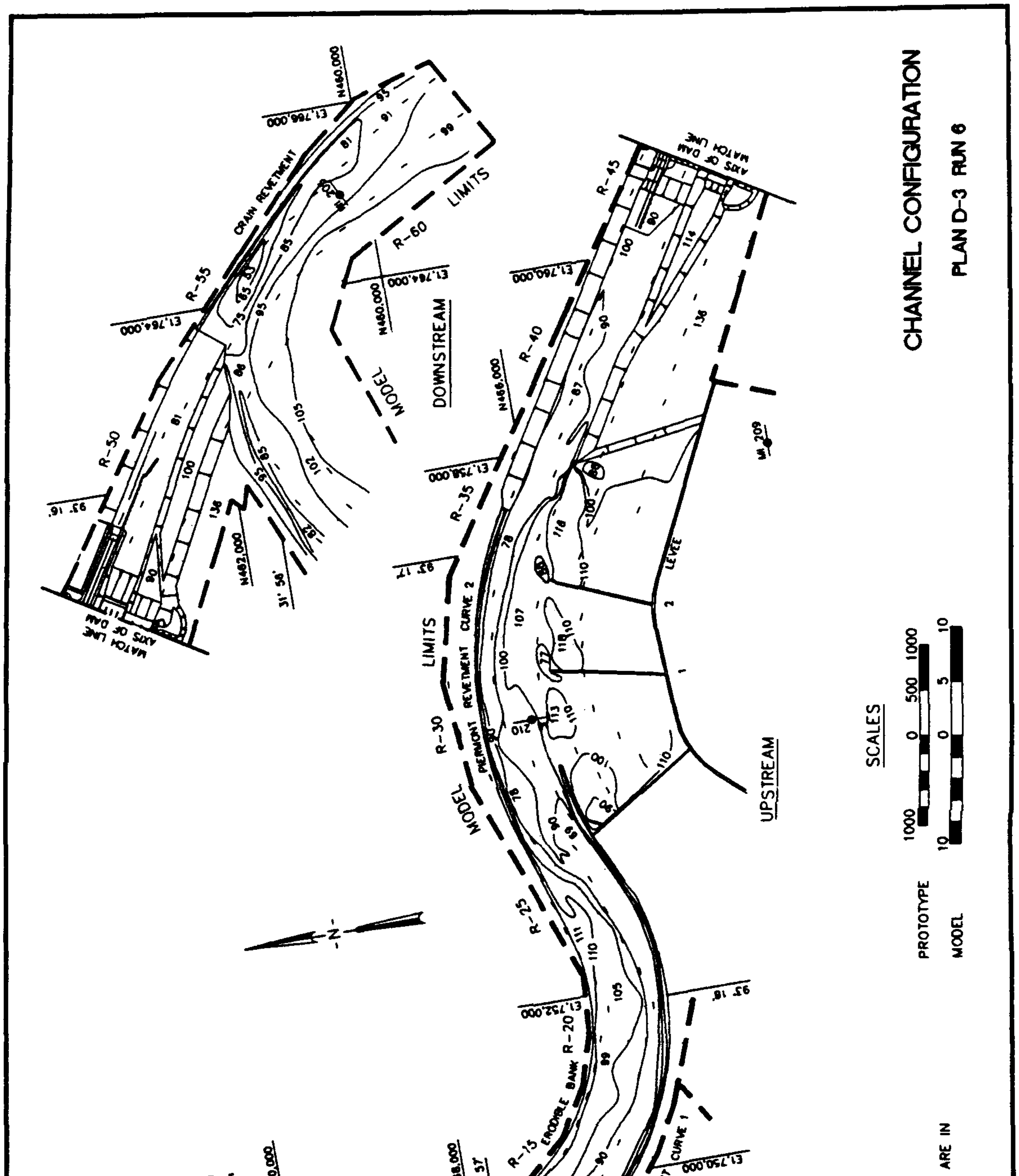

CHANNEL CONFIGURATION
PLAN D-3 RUN 5
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
MODEL LIMITS
DOWNSTREAM
UPSTREAM
LEVEE
BULL REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
CRAIN REVETMENT
MATCH LINE
AXIS OF DAM
MI 212
MI 209
R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
R-55
R-60
N472,000
N470,000
N468,000
N466,000
N462,000
N460,000
E1,748,000
E1,750,000
E1,752,000
E1,758,000
E1,760,000
E1,764,000
E1,766,000

PLATE 40


CHANNEL CONFIGURATION
PLAN D-3 RUN 6
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
UPSTREAM
DOWNSTREAM
MODEL LIMITS
BULL REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
CRAIN REVETMENT
LEVEE
AXIS OF DAM
MATCH LINE

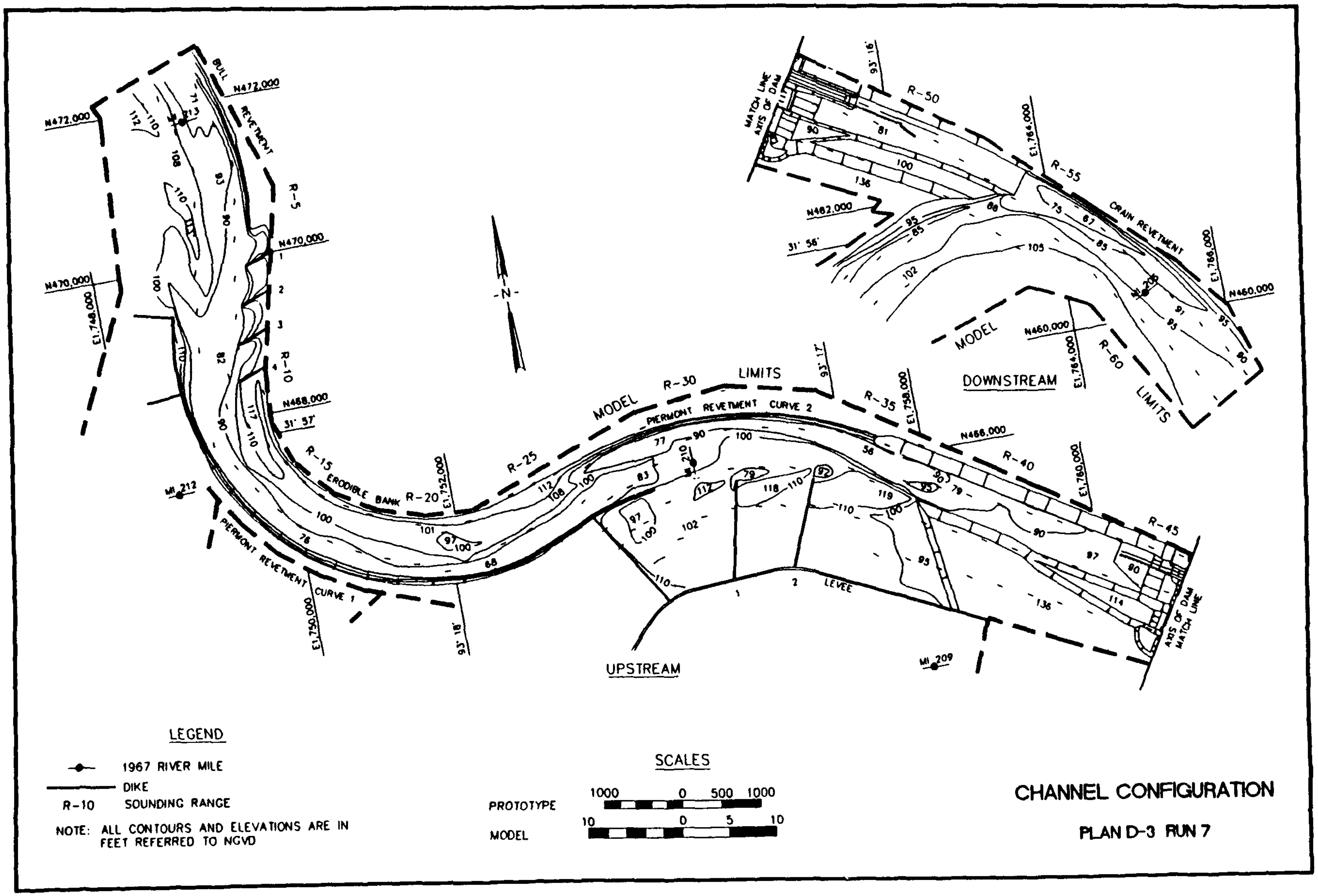
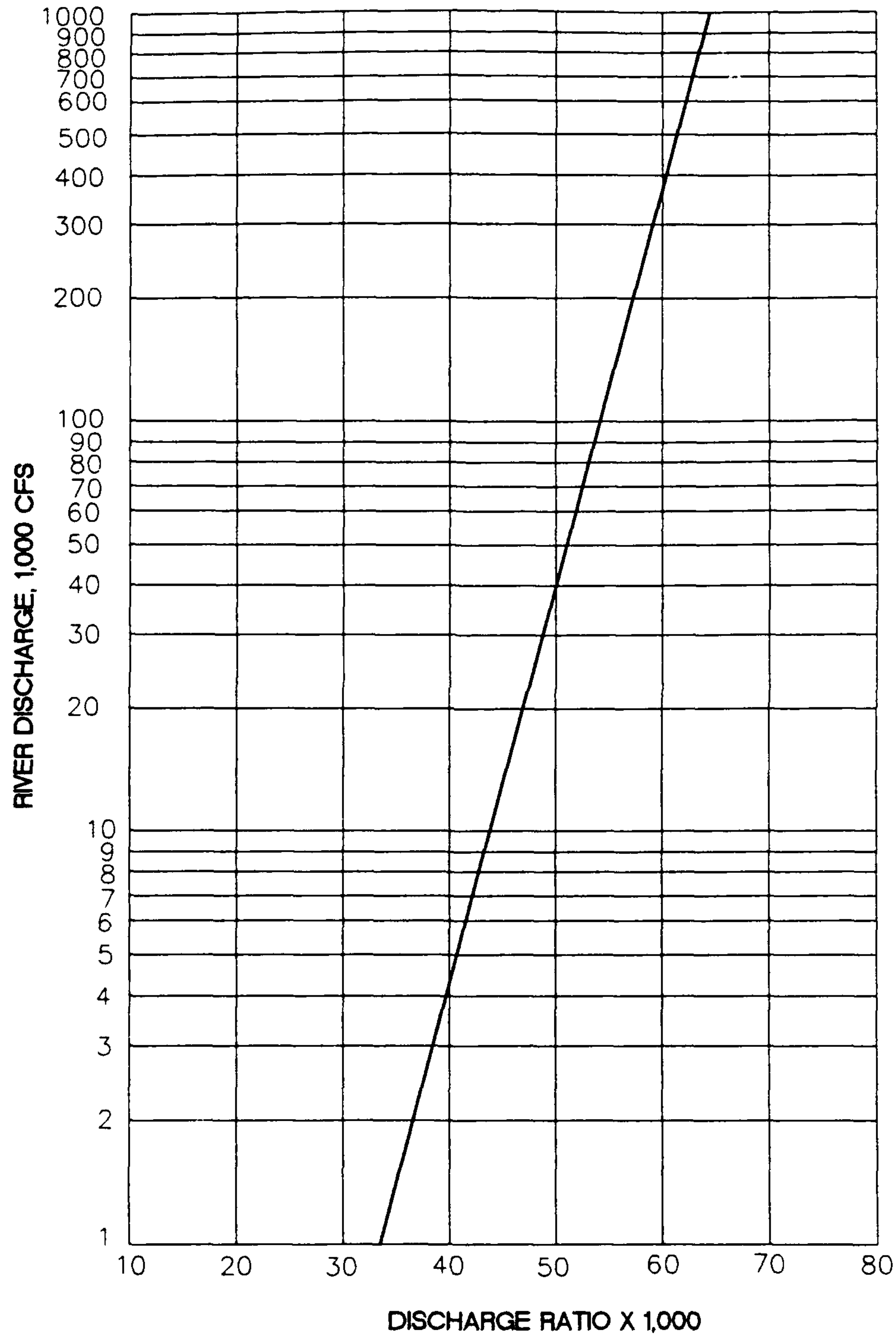

CHANNEL CONFIGURATION
PLAN D-3 RUN 7
LEGEND
1967 RIVER MILE
DIKE
R-10 SOUNDING RANGE
NOTE: ALL CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO NGVD
SCALES
PROTOTYPE 1000 0 500 1000
MODEL 10 0 5 10
UPSTREAM
DOWNSTREAM
MODEL LIMITS
BULL REVETMENT
PIERMONT REVETMENT CURVE 1
PIERMONT REVETMENT CURVE 2
ERODIBLE BANK
CRAIN REVETMENT
LEVEE
MATCH LINE AXIS OF DAM
-N-
R-5
R-10
R-15
R-20
R-25
R-30
R-35
R-40
R-45
R-50
R-55
R-60
N472,000
N470,000
N468,000
N466,000
N462,000
N460,000
E1,748,000
E1,750,000
E1,752,000
E1,758,000
E1,760,000
E1,764,000
E1,766,000
31° 57'
31° 56'
93° 18'
93° 17'
93° 16'
MI 212
MI 210
MI 209

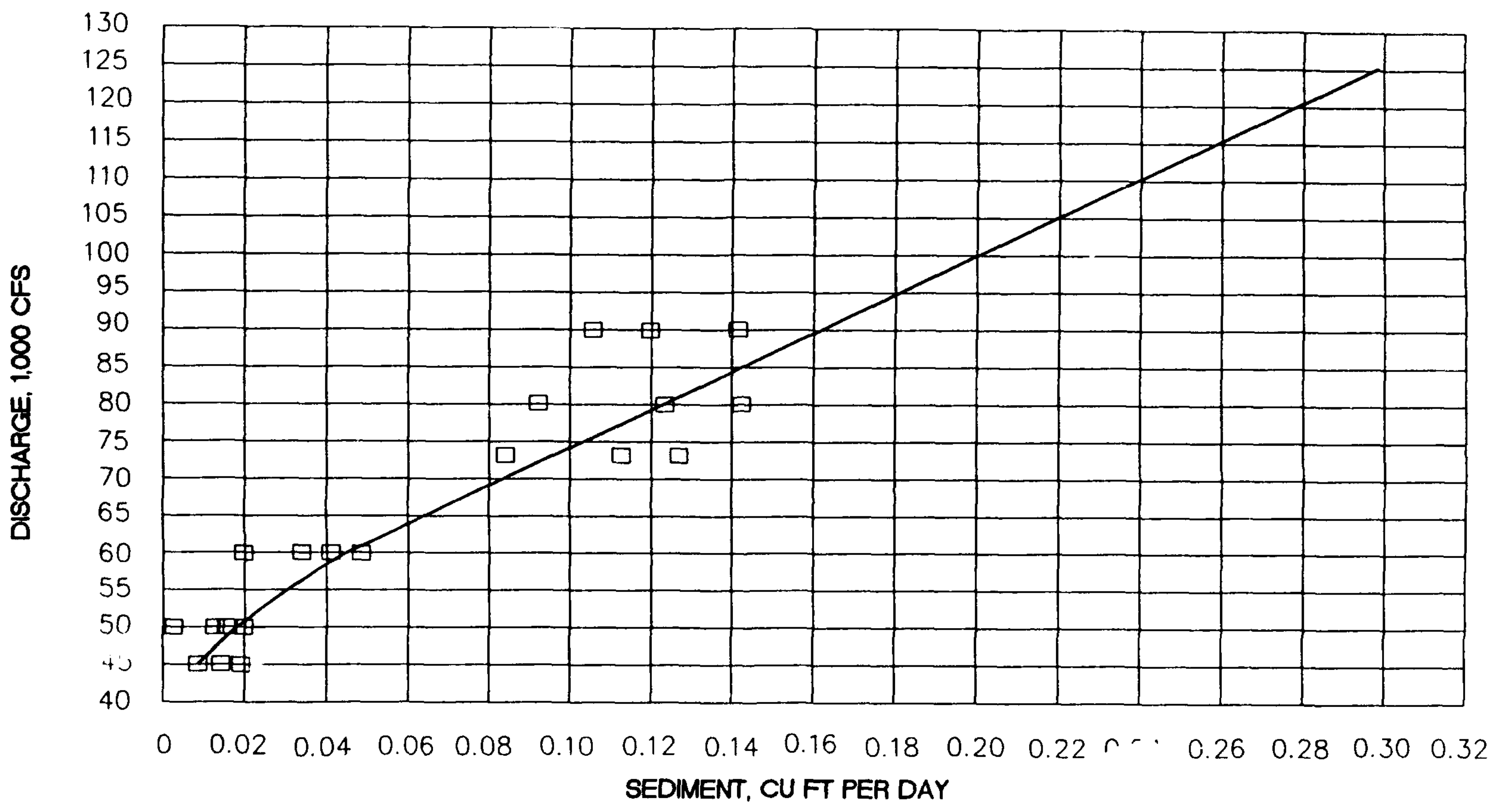
PLATE 42

APPENDIX A: MODEL VARIABLES



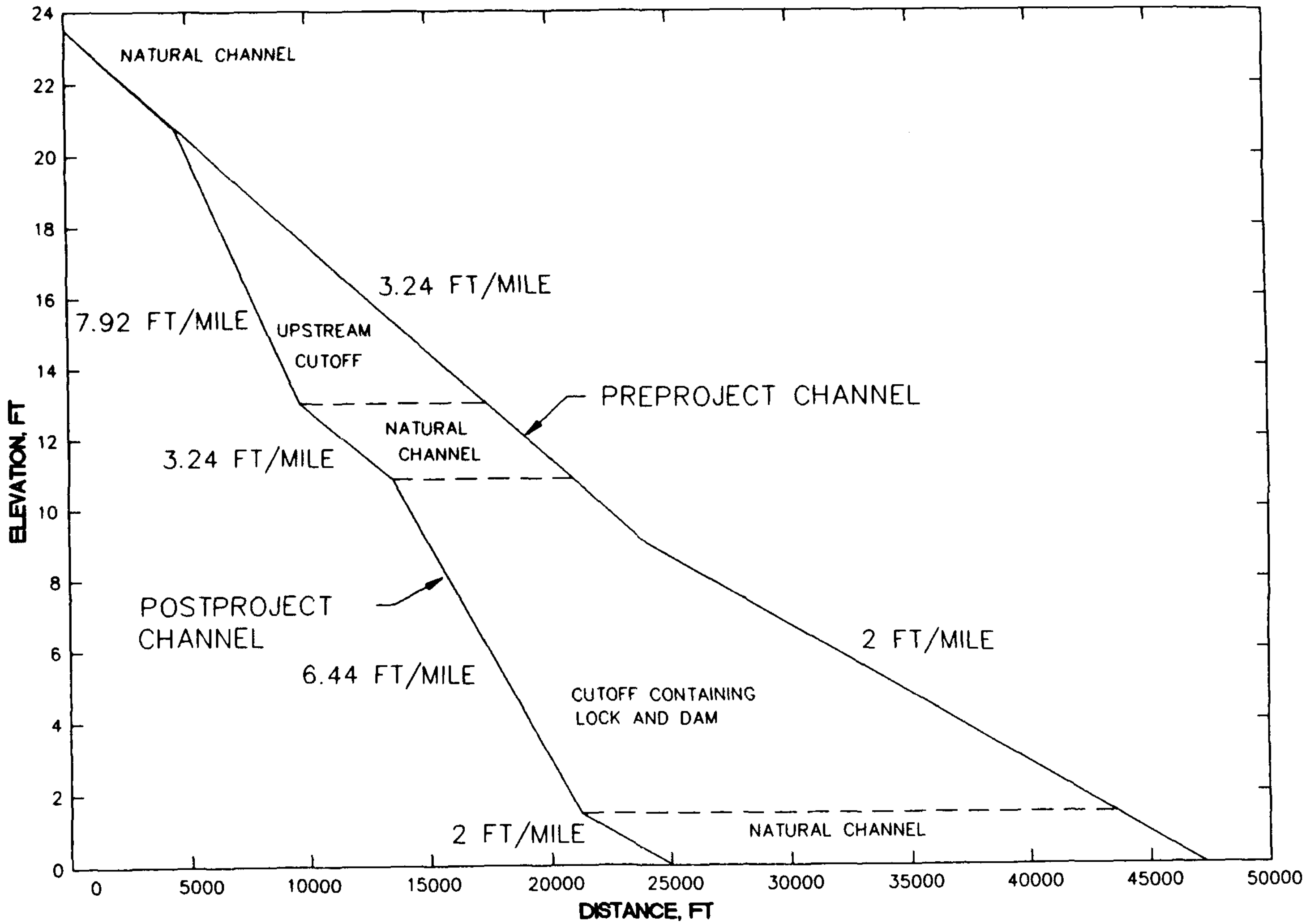
DISCHARGE RATIO CURVE

PLATE A1



SEDIMENT INPUT CURVE

PLATE A2



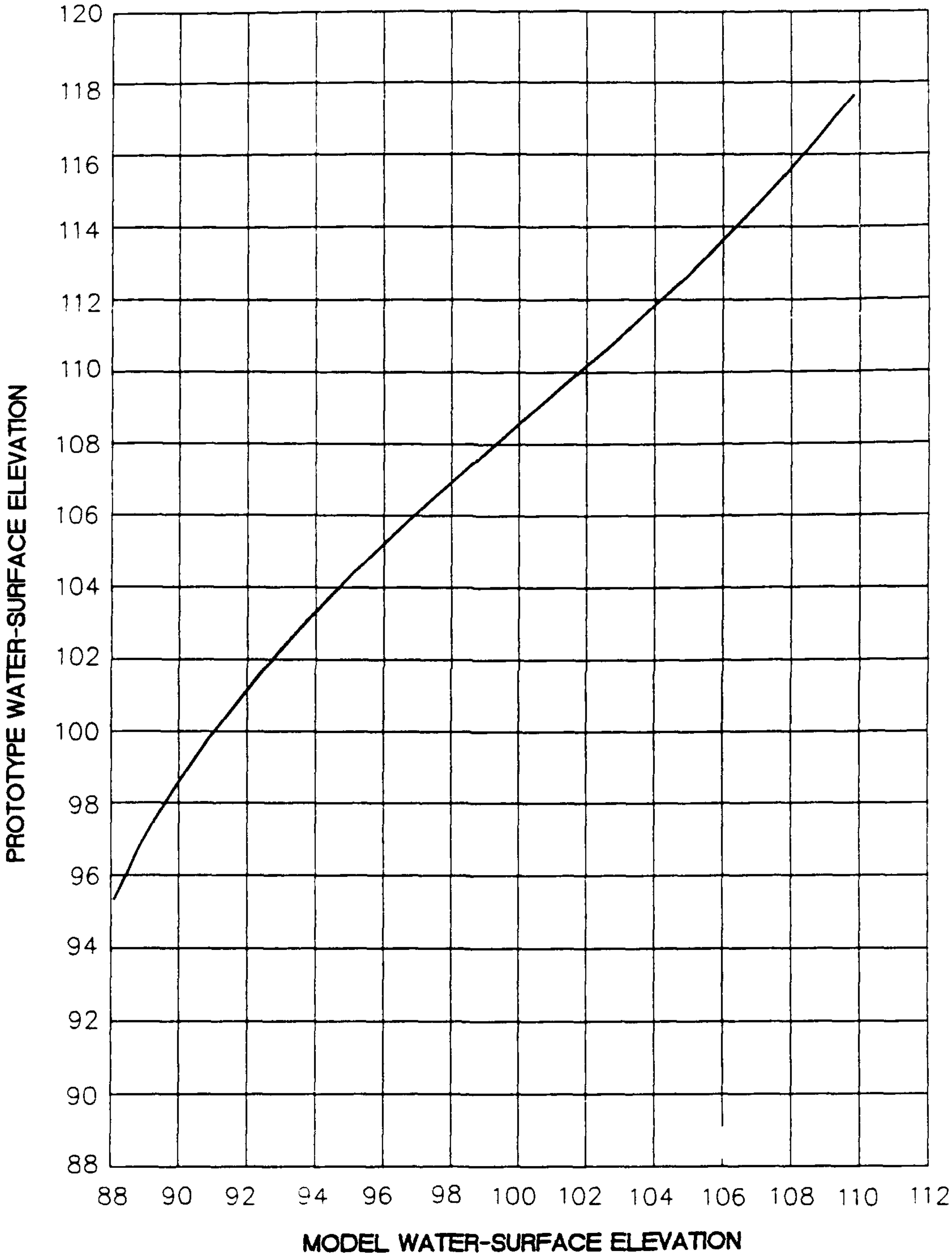
SUPPLEMENTAL SLOPE

APPENDIX B: MODEL TO PROTOTYPE STAGE CONVERSIONS

1. The distance through the cutoff to the location of the headwater control was computed as a percentage of the length of the whole cutoff. This percentage was then applied to the natural channel and resulted in a location at 1967 river mile 207.27. Model data were available at 1967 river miles 207 and 208, model gages 8 and 7, respectively, as shown in the following tabulation. The prototype data were obtained from the revised HEC-2 model. Plate B1 shows the prototype to model conversion.

Gage Description	1967 River Mile	Prototype Stage	Model Stage
Model Gage No. 7	208	120.0	119.0
Headwater (control)	207.27	120.0	117.9
Model Gage No. 8	207	120.0	117.5

2. During model operation the headwater was maintained at el 120 until the flow was great enough to cause an open river condition. During open river flow the stages are controlled using the last gage on the model and the headwater gage readings are collected for information only. Therefore, only el 120 needed to be converted to a model stage to allow headwater control.



PROTOTYPE TO MODEL CONVERSION
1967 RIVER MILE 204.8

Waterways Experiment Station Cataloging-in-Publication Data

Mueller, David S.
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1. Red River (Tex.-La.) — Navigation. 2. Sediment transport — Red River (Tex.-La.) 3. Locks (Hydraulic engineering) — Louisiana. 4. Stream channelization. I. Title. II. Maggio, David M. III. Pokrefke, Thomas J. IV. United States. Army. Corps of Engineers. Vicksburg District. V. U.S. Army Engineer Waterways Experiment Station. VI. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-90-2.
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